DEVELOPMENT AND TESTING OF RAPID REPAIR METHODS FOR WAR DAMAGED RUNWAYS

BY

PAUL A. SOARES

A REPORT PRESENTED TO THE GRADUATE COMMITTEE
OF THE DEPARTMENT OF CIVIL ENGINEERING IN
PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR TH'. DEGREE OF MASTER OF ENGINEERING

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DEDICATION

This report is dedicated to my beautiful wife, Kathleen, for all of her loving support through these two years of postgraduate school.

ACKNOWLEDGEMENTS

Sincere gratitude is extended to Dr. David Bloomquist for all of his assistance and guidance throughout the entire length of this research in determining and verifying the mix design procedures. Spacial acknowledgement and thanks is also extended to Professor Will Shafer and Doctor Ralph Ellis for their assistance and guidance in preparing this report.

A special thanks to Dr. Fagundo for assisting in the development and evaluation of the initial ideas of this research. The author wishes to express appreciation to Dr. Byron E. Ruth for his interest and guidance in developing mix design approaches.

The author extends special gratitude to Mr. Charles Allen for providing vital information on aggregate properties vital to performing this research.

The author sincerely thanks Mr. Daniel Richardson for his super assistance in the Civil Engineering Materials Laboratory.

The author acknowledges the integral assistance of the Florida Department of Transportation, especially to Mr. Toby Larson and Dr. Armaghani

A special thanks to Mr. Richard Coble for working in conjunction with me to provide knowledge of construction equipment capabilities and procedures

as well as actual construction equipment, materials, skilled equipment operators, a testsite, and funding needed to complete the field test.

PREFACE

The problem of effecting repair to war damaged runways within the time constraints of a combat situation currently looms at large for the Armed Services of the United States. Although several methods have been proposed and tested, none have been adopted as the answer to the problem. Research continues on in earnest at several Department of Defense Research Centers.

As a Naval Officer with 2.5 years of hands-on experience in effecting repair to war damaged runways, I chose this subject as the basis for my Master's Report. After consulting with several prominent professors and engineers, I had gathered several solutions to the problem. One concept that looked very promising was conceived through consultation with Mr. Richard Coble.

Mr. Coble is attending the University of Florida to obtain a PH.D. in Architecture. Mr. Coble is president of KACO Construction Company which performs approximately 15 million dollars worth of construction annually. Mr. Coble has a Bachelor of Science Civil Engineering and Master's Degree in Building Construction.

The concept involves mixing the materials in place. My work began with developing a mix design utilizing roller compacted concrete technology. I

also formed rough ideas as to how the method could be developed into a construction technique.

With my construction ideas, I consulted with Mr. Coble as to the validity of the techniques. Due to his 25 yrs of contracting experience, with emphasis in equipment and soil stabilization, he was able to evaluate my ideas and develop additional ideas to form a construction method.

At this point, I looked toward development of a lab test on the smallest scale possible, that would still simulate the actual conditions of the real scenario of repairing war damaged runways. I would face the limitations of how close the simulation would come to the real construction operation.

Upon further consultation with Mr. Coble, an agreement was reached to conduct a full scale test as a joint research effort between myself and Mr. Coble. The joint effort would involve the pooling of my experience as Naval Officer working in war damage repair and equipment management, along with Mr. Coble's experience as a general contractor specializing in equipment management.

The test would have the resources of Mr. Coble's Company, KACO Construction, at its disposal including a test site to perform the full scale test. The resources included all necessary equipment,

operators, materials, computers, video equipment, and other resources that would not have been otherwise available.

Therefore, through the teaming of my naval experience and engineering knowledge along with Mr. Coble's experience in equipment capabilities, construction techniques, and the resources of KACO Construction Company, the research progressed at a pace which completed all design, development, testing, and verification of results on a large scale test within a four month timeframe.

The successful completion of this research can only be attributed to the diverse backgrounds of each partner coming together to form a competent team. Each partner obtained vital information from several sources including prominent professors and engineers.

The preparation of this report covers the work completed by myself for the fulfillment of the requirements of my master's degree. The information and materials obtained through Mr. Coble, along with all other sources, are noted in the reference section.

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CHAPTER 1
INTRODUCTION

1.1 Problem Statement

In a combat/hostile scenario, the USAF main operating bases (MOBs) and U.S. Naval Air Stations (NAVAIRSTA) must function efficiently and effectively despite runway damage. The MOBs must support tactical aircraft launch and recovery (1). The NAVAIRSTAs must provide critical logistics support to the fleet, forward deployed antisubmarine warfare (ASW) aircraft bases, and alternative launch and recovery sites for carrier aircraft (2). Because of the important functions served by MOBs and NAVAIRSTAs, they will become primary targets for enemy and/or terrorist forces. The damage sustained from enemy and/or terrorist forces must be repaired in a rapid and effective manner.

The term rapid is typically interpreted to mean within 24 hrs, however, many cases will require repair in less than 24 hrs (i.e. returning aircraft that cannot be diverted). The term damage is typically interpreted to include craters and spalls (small holes not completely through the slab).

The current rapid repair methods include using crushed stone with some type of cover to prevent loosened stones from entering jet engine intakes to cause foreign object damage (FOD) to the engine turbines. The types of FOD covers are Fiber reinforced Polyester (FRP) mats, AM-2 matting, precast concrete slabs, and quickset concrete utilizing a Cretemobile to batch the concrete (3).

Each type of FOD cover has unique disadvantages. The FRP mats are constantly being researched to improve their anchoring capability. The significant problem is that the mats do break loose from their anchoring due to lateral forces imparted onto ths mats by high speed contact with aircraft landing gear. Sliding FRP mats could cause damage to aircraft landing gear during landings. Also, loose stone escapes from areas of the backfilled crater that are exposed due to the sliding mat. The FRP mats do not provide any structural support for the crushed stone, therefore, the crushed stone base ruts from aircraft traffic creating a need for the FRP mats to be removed for routine leveling and compacting of the crushed stone base.

The AM-2 matting is an interlocking steel plate system. A plate can be placed manually by two men. The 1.5" thickness of the plates causes a jolt to aircraft landing gear when they traverse from existing pavement onto the AM-2 matting. The jolt is beyond the cyclic loading tolerance for landing gear of the Navy P-3, Air Force F-15, and F-16 aircraft (4). The aircraft type restriction renders AM-2 almost useless for main runway repair.

Precast concrete slabs are placed by construction equipment. The slabs are typically two meter squares with a depth of 6 inches (5). The precast slabs typically undergo differential settlement from slab to slab. The time required for cutting old concrete and heavy equipment required for placement of slabs along with differential

settlement makes the precast slab method undesirable for main runway repair.

The use of quickset concrete requires time to attain an initial set from the plastic state of wet concrete. The quickset concrete method also requires the use of a Cretemobile which is a truck with a small batch plant capable of producing approximately 25 CY/hour (6). The output is directly controlled (critical path) by the number of Cretemobiles onsite. The initial time of non-use, due to the plastic state of the wet concrete, and the limited Cretemobile output makes the quickset concrete method difficult to utilize as a main runway repair method.

A need exists to establish a main runway repair method that is rapid, effective, and easy to install.

1.2 Study Objectives

The primary objective of this research was to develop a rapid runway repair method that could be implemented by the armed forces within their current resources.

1.3 Scope of Work

A method of mixing fine aggregate, coarse aggregate, cement, and water was developed using soil stabilizing equipment.

The required testslab thickness was selected from known slab thicknesses in existence today. The mix design was developed for a roller compacted mix. The mix design was tested in a laboratory for compressive, tensile and flexural strength. Simplified relationships were developed between over

the degree of compaction versus the strength of concrete.

Moist Rodded Unit Weight tests were performed on the fine and coarse aggregates to determine a pre-mix unit weight of each material. The mix proportions by weight of fine aggregate, coarse aggregate, and cement were converted to pre-mix volumes. An angle of repose test was performed on the fine and coarse aggregates. The pre-mix volumes were converted to layer thicknesses.

A field test procedure was developed to conduct a large scale test of the method. A large scale test was performed.

Settlement tests were conducted on an hourly basis to determine the rate of settlement versus time relationship. Test specimens of concrete material were collected and tested for compressive strength.

Flexural and tensile strengths were calculated using known conservative compressive to flexural strength relationships. Analysis was performed to determine the degree of compaction versus the strength relationships at different times of curing.

Conclusions were drawn as to the overall suitability of this method for main runway repair. Finally, several recommendations were made for areas were further research could be conducted for this method.

Keywords: settlement tests, flexural strength, tactical strength, (KR

CHAPTER 2
FIELD TEST PROCEDURE

2.1 Construction Sequence

2.1.1 Master Activity Listing

The critical path activities were developed through consultation with Rick Coble, President of KACO construction company (7).

The critical path activities in the construction sequence were as follows:

1. MARK-OFF HOLE; Mark-off equivalent hole
2. PLACE COARSE LAYER; Fill-in and level off coarse layer
3. PLACE CEMENT; Lay cement bags down at proper intervals. Open cement bags and spread out powder in thin layer. Place small amount of sand simultaneously on top of cement powder.
4. PLACE FINE LAYER; Place sand on top of cement and level off to desired height.
5. BLEND DRY MIX; Mix aggregates and cement together with soil stabilizer.
6. FORM LIP AND GROOVES; Form lip around mixed materials and make grooves in material perpendicular to the slope of existing ground.
7. APPLY WATER; Apply water by watertruck.
8. WINDROW EDGES IN; Windrow edges of layers towards center of layers with a motor-grader.
9. BLEND WET MIX; Mix water, aggregates, and cement together with mixer.

10. FILL HOLE; Blade material into hole with grader

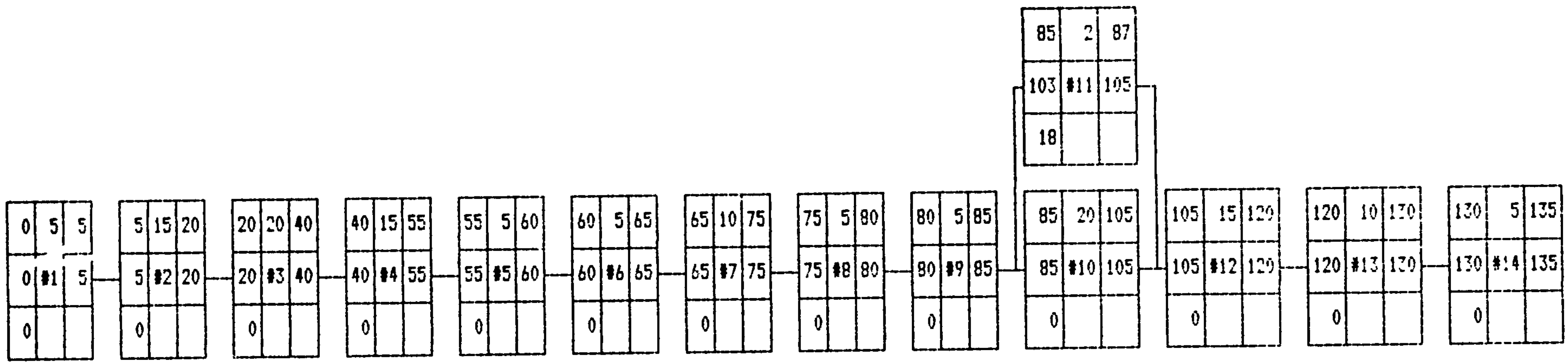
11. WET DRY SPOTS; Apply water with a hose in small quantities as grader blades material into hole.

12. COMPACTION; Compact with roller.

13. REMOVE EXCESS; Grade off excess with grader.

14. FINISH ROLL; Make final roller pass to obtain smooth finish

The CPM diagram is shown in Figure 2.1.



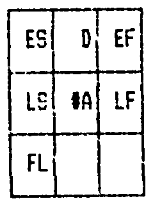
MASTER ACTIVITY LISTING

1. Mark-Off Hole
2. Place Coarse Layer
3. Place Cement
4. Place Fine Layer
5. Blend Dry Mix
6. Form Lip and Grooves
7. Apply Water
8. Windrow Edges In
9. Blend Wet Mix
10. Fill Hole
11. Wet Dry Spots
12. Compaction
13. Remove Excess
14. Finish Roll

CREW SIZE: 1 Foreman
2 Equipment Operators/Mechanics
1 Laborer

CRATER SIZE: 30 FT * 9 FT * 10 IN

LEGEND



ES - Early Start
D - Duration
EF - Early Finish
LS - Late Start
#A - Master Activity Number
LF - Late Finish
FL - Float
Time in Minutes

FIGURE 2.1 CPM DIAGRAM: FIELD TEST PROCEDURE

2.1.2 Construction Sequence Conditions

The conditions before performing the first master activity were:

1. The layer depths were already calculated. The base layer width and length were calculated. More discussion on layer dimension calculations is contained in chapters 3 & 4.

2. The crater was backfilled to a uniformd depth of 8 inches and compacted.

3. The area surrounding the crater was paved.

2.1.3 Construction Sequence Discussion

All photographs used as figures in this section were taken and developed by Rick Coble of KACO construction company (8). Through the use of a stringline, an area to place the material was marked off that was outside of the crater. This marked off area or box is shown in Figure 2.2.

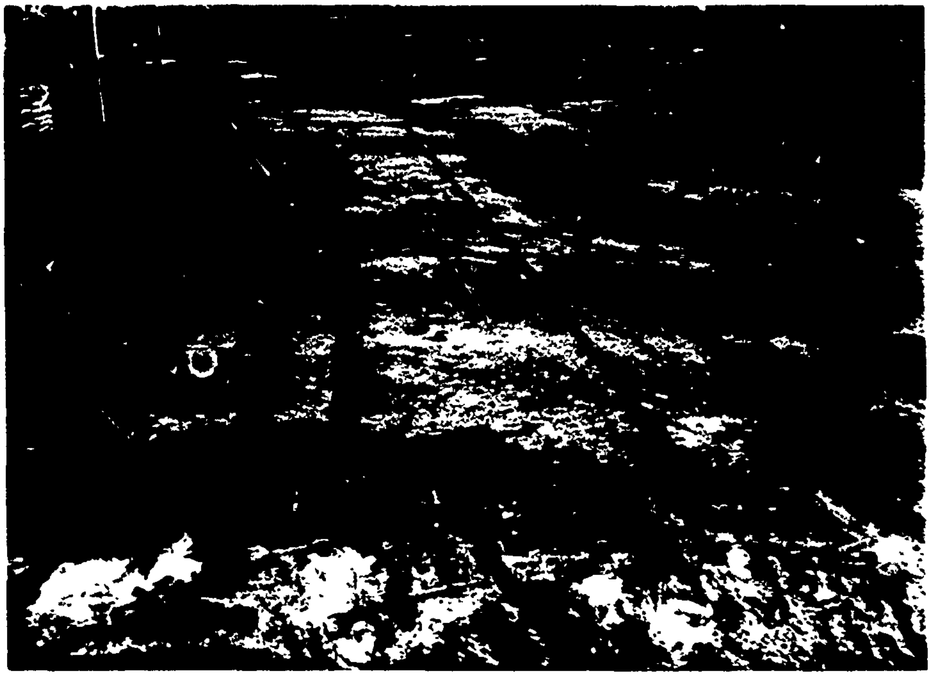


Figure 2.2 Marked Off Area (Equivalent Hole)

The coarse aggregate was placed first into the marked off box. The use of the marked off box to place the material in rather than placing the material directly into the hole is called the equivalent hole concept. Further discussion on the equivalent hole concept is contained in chapter 4. The keypoint to understand at this point is that the material placed above ground in the marked off box was the amount plus waste required to fill the testhole. Figures 2.3 and 2.4 show the coarse aggregate filling operation. The coarse aggregate was placed forming a trapezoid up to a depth of 5.25 inches (approximately equal to exact calculated depth of 5.3 inches). Figures 2.5 and 2.6 show the measuring operation. The layer depth calculations for all layers are contained in chapter 4.

Figure 2.3 Partial Coarse Aggregate Placement

Figure 2.4 Front End Loader Placing Coarse Aggregate

Figure 2.5 Leveling Off Coarse Aggregate

Figure 2.6 Measuring Coarse Aggregate Layer

The cement bags were spaced apart at 2 ft 6 inches as shown in Figure 2.7. The cemsnt bag spacing calculations are covered in section 4.4. The cement bags were opened and spread by hand. Small amounts of sand were simultaneously placed on top of the cement powder to prevent the wind from blowing away the cement powder. Figure 2.8 shows the cement and sand placing operation at completion.

Figure 2.7 Cement Bag Placement

Figure 2.8 Cement Powder Placement With Sand Cover

Place fine aggregate (sand) layer up to desired height. Figure 2.9 shows sand leveling off operation.

Figure 2.9 Sand Leveling Off and Measuring

The soil stabilizer was then positioned at one end of the layers as shown in Figures 2.9 & 2.10. The mixing action was accomplished by the mixing tines as shown in Figure 2.11.

Figure 2.10 Soil Stabilizer Placement

Figure 2.11 Soil Stabilizer Mixing Tines

The mixing was completed in one pass, Figure 2.12 shows the soil stabilizer performing the mixing operation.

Figure 2.12 Dry Mixing of Aggregate and Cement Powder

A lip was formed around the mixed materials, as shown in Figure 2.13, to retain the following addition of mixing water.

The water is added by means of a spray bar connected to the watertruck as shown in Figure 2.14. The calculations for the flowrate of the water and the velocity of the truck are covered in a following section in this chapter.

Figure 2.13 Lip Formation

Figure 2.14 Water Application

A field discovery and modification of the technique was the scoring of the surface of the layers. A scoring technique must be used to make grooves in the material perpendicular to the slope of existing ground. This will allow water to reach the lower depths of the mixed materials. Otherwise, the material becomes saturated near the top only before becoming impermeable due to water surface tension causing tight adhesion between particles. This temporary impermeability will cause some mixing water to run off of the layers onto the surrounding ground, as shown in Figure 2.15, thereby, leaving mixed material at lower depths completely dry.

A deviation from the actual procedure used in the field will be recommended here. It is recommended that a motor-grader be used, before the first mixing, to windrow the edges of the layers back into the center of the layers. This is recommended because the edges are typically dry due to the necessary retention of water towards the center of the layers by the containment lip to prevent water run-off. Once the pooled water seeps into the lower center of the mixed material layer, the dry edges should be windrowed into the center to allow improved mixing by the soil stabilizer. During the field test the wet material was mixed first and then the edges were windrowed in afterwards as shown in Figure 2.16. This necessitated an extra pass by the soil stabilizer to mix in dry edges windrowed into the center.

Figure 2.15 Water Run-Off

Figure 2.16 Grader Windrowing Wet Material

The soil stabilizer was positioned and then run over the wet windrowed material as shown in Figures 2.17 & 2.18.

Figure 2.17 Repositioning Soil Stabilizer

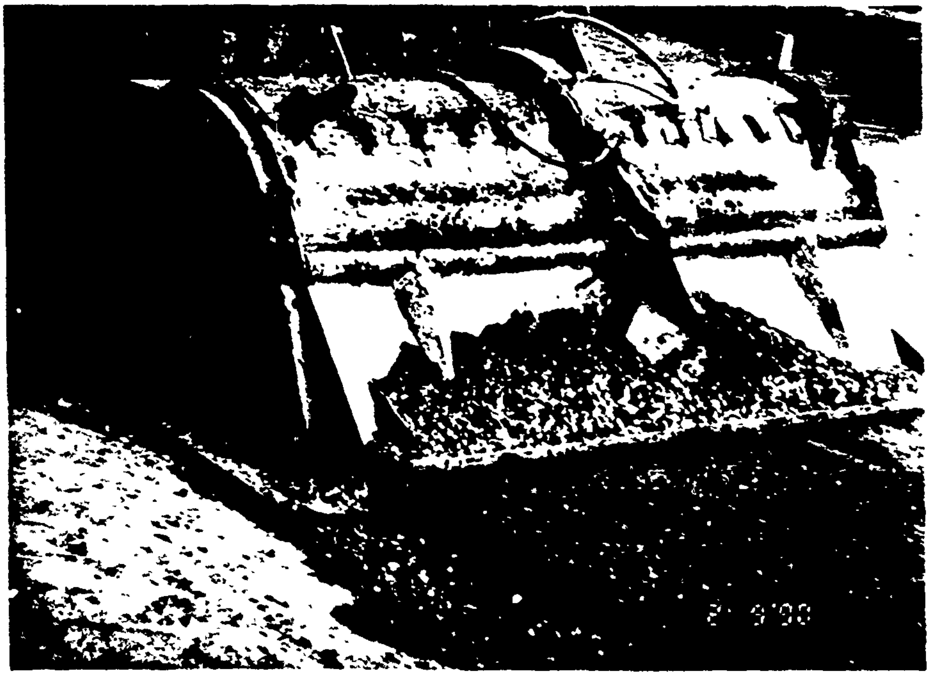


Figure 2.18 Wet Mixing of Aggregate and Cement Powder

The motor-grader was used to blade the material into hole as shown in Figures 2.19 & 2.20. This provided extra mixing by windrowing the material in addition to transporting the material into the hole.

Figure 2.19 Grader Windrowing Wet Mixed Material

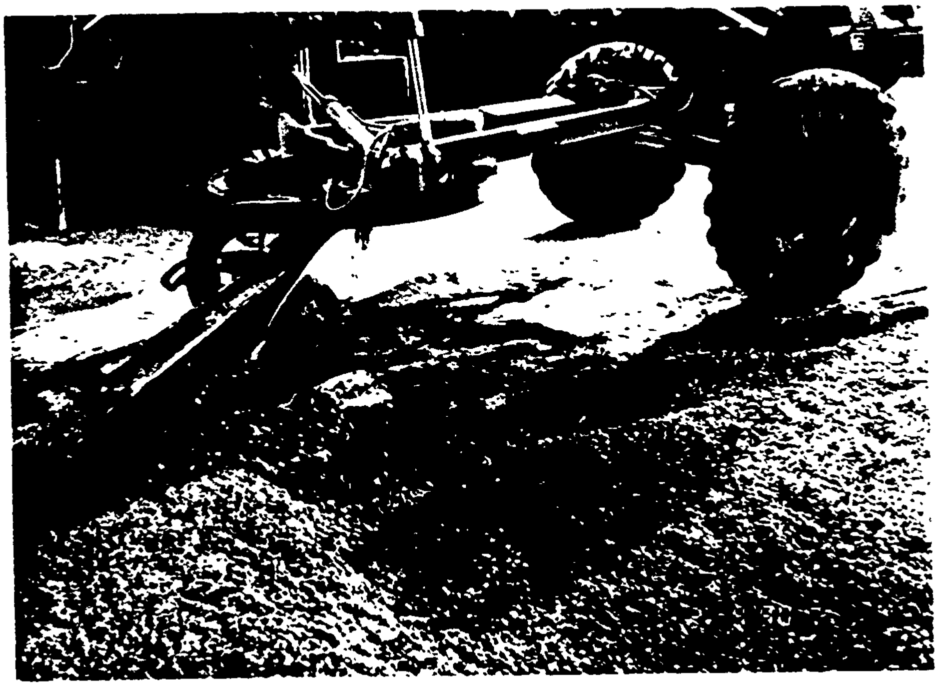


Figure 2.20 Grader Windrowing Wet Mixed Material

Water was added to exceptionally dry areas by means of a hose as shown in Figure 2.21. Further discussion concerning permissible quantities of water that can be added by hose are discussed in following sections in this chapter. The addition of water was performed while the motor-grader was blading the material into the crater.

The material was compacted by means of a smooth drum vibratory roller. Several passes were made over the material as shown in Figures 2.22 & 2.23. The total time of compaction was approximately 15 minutes. The material deformed slightly (less than 1 inch) under vibratory compaction as shown in Figure 2.24. Overall, the material exhibited very good stiffness.

The material that remained above the desired elevation of the slab was graded off using the motor-grader.

One pass with the roller was made with no vibration to smooth out the final finish of slab.

Figure 2.21 Additional Water Placement

Figure 2.22 Vibratory Roller and Entire Slab View

Figure 2.23 Vibratory Roller and Partial Slab View

Figure 2.24 Vibratory Roller (Close-Up View)

2.2 Water Application

The calculations for the application of water to the dry mixed layers will be discussed in this section. The first step was the calculation of flowrate.

2.2.1 Determining Flowrate

To determine the flowrate of the watertruck, the water truck was backed up to a trapezoidal container. Two trials of filling the containers were performed. In trial 1, the spray bar was placed over the container, the water was turned on and allowed to fill the container as shown in Figure 2.25. The water was turned off without moving the truck. In trial 2, the truck was driven off to discontinue water flow into the container as shown in Figure 2.26. The time required to fill the container in each trial was measured and recorded. The volume of water in the filled container was calculated by measuring the depths as shown in Figure 2.27. The volume calculations are contained in Appendix A.

The volumes of water and times were recorded for two trials. The average of the two volumes and times were used to calculate the flowrate. Table 2.1 shows the values recorded.

Table 2.1 Water Volumes and Times

	Volume	Time
Trial #1	49.00 gal	25.40 sec
Trial #2	50.78 gal	26.00 sec
Average	49.89 gal	25.70 sec

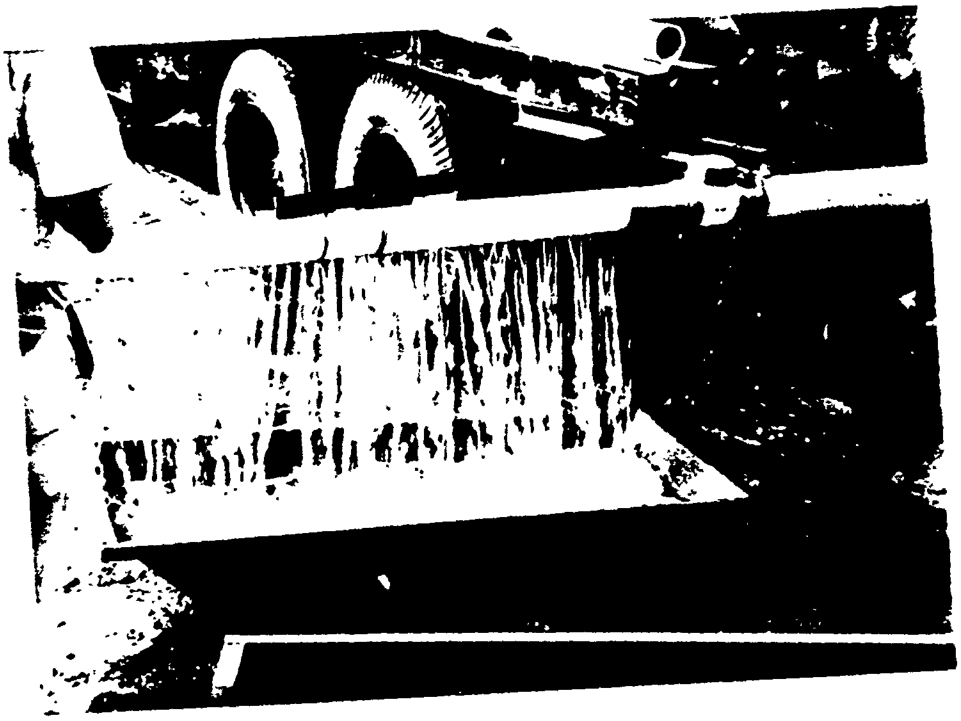


Figure 2.25 Water Container Filling (Trial 1)

Figure 2.26 Water Container Filling (Trial 2)

Figure 2.27 Depth Measurement

The flowrate was calculated by means of equation 2.1.

$$\text{Flowrate} = \frac{\text{Volume of Water}}{\text{Time}} \qquad (2.1)$$

$$\text{Flowrate} = \frac{49.89 \text{ gal}}{25.70 \text{ sec}} = 1.941 \text{ gal/sec}$$

If the watertruck will be emptied out by placing the water, then a change in head will occur as the tank empties This variance in head must be accounted for by recording the flowrate at various heads within the watertruck tank and calculating an average flowrate. If the variance is significant then two or more averages may be required to keep the flowrates within an acceptable margin of error. The watertruck level will then have to be watched to determine when flowrates and therefore truck velocities need to changed. The allowable flowrate margin of error is discussed in the sensitivity analysis chapter in this report. The watertruck used in the test had a 1200 gal capacity. Therefore, the water required was only a quarter of the truck capacity thereby causing no significant change in flowrate.

2.2.2 Determining Velocity

Once the flowrate was obtained, the velocity of travel of the truck could be calculated. The total quantity of water required was calculated based on the mix design and total volume of concrete mix to be created.

Establishing parameters for the watertruck was the next step. The spray bar width was easily adjusted by sealing

off the small holes in the PVC pipe with tape as shown in Figure 2.25. The width of the layers was determined based on the optimum mixing width of the soil stabilizer. Therefore, the water spray bar width was set to that optimum mixing width of 5 feet. The spray bar width must equal the least width or some multiple of the least width of the crater.

The reason for using the least width criteria rather than average width was that the water would leach into the existing ground or run-off when passing over narrow portions of the crater. Therefore, the volume of water placed into the layers would be reduced.

If there is a great variance in width due to irregular craters or circular craters, then the use of an equivalent hole is recommended as discussed in chapter 4. Figures 4.1 & 4.3 show layouts of the equivalent hole concept on irregular or circular craters. The parameters discussed so far are contained in Table 2.2 below.

Table 2.2 Watertruck Parameters

Sprayer Width	= 5 ft
Measure Least Width of Crater	= 5 ft
Crater Least Width/Spray Width	= 1 ft

The required number of passes over the material is a function of the crater least width to sprayer width ratio as shown above in Table 2.2. When the ratio is one, the required number of passes equals one. Another factor that

can affect the required number of passes is the low velocity required by the truck to allow enough time to place all of the required water. This was the situation that occurred during the field test. The required number of passes was set at 2 as shown in the following calculations.

The parameters for the testhole are shown below in Table 2.3 below.

Table 2.3 Testhole Parameters

Single Pass Driving Distance	= 90.0 ft
Total Driving Distance	= 180.0 ft
Volume of Water Needed	= 215 gal

The single pass driving distance was equal to the length of the laid out material. The number of passes was set at 2, therefore the total driving was equal to 180 ft. The volume of water needed is listed in Table (4.4) at 1791 lbs which was converted to gals.

The watertruck velocity was calculated by means of equation 2.2.

$$\text{Watertruck Velocity} = \frac{\text{Flowrate}}{\text{Water Volume}} * \text{Total Driving Distance} \qquad (2.2)$$

$$\text{Watertruck Velocity} = \frac{1.941 \text{ gals/sec}}{215 \text{ gals}} * 180 \text{ ft}$$

Watertruck Velocity = 1.625 ft/sec

The total driving time was calculated by equation 2.3.

$$\text{Total Driving Time} = \frac{\text{Total Driving Distance}}{\text{Watertruck Velocity}} \qquad (2.3)$$

Total Driving Time = 111 sec

The conversion to miles per hour (1 MPH = 1.467 feet/sec) yields a speed that cannot realistically be driven by attemptin to read a speedometer as shown below.

Watertruck Velocity = 1.110 mph

If the number of passes had been left at 1, then the required watertruck speed would have been halved to 0.555 mph which was too slow for the truck idle speed. Therefore, the slowness of the required velocity at one passage necessitated 2 passes to increase the watertruck velocity.

Measuring a velocity of 1.11 mph by speedometer would also be difficult. Therefore, in order to spray the amount of water required with accuracy, the velocity was measured as discussed in the following section.

2.2.3 Measuring Velocity

The velocity of the watertruck was controlled by timing the watertrucks passage over the material. By keeping the watertruck's passage to the required driving time, the truck velocity was controlled. Getting the exact required time of passage required a few dry runs to allow the watertruck operator to gauge the truck's speed more accurately. The actual rus were then timed by one person who talked to the operator as he drove as was shown in Figure 2.15.

2.2.4 Foul Weather Procedure

The onset of foul weather presents obvious problems to the application of the proper amount of water to the material. The logic to overcome this problem depends on the current stage of construction when the foul weather occurs.

If the cement powder has not been placed, then there are two options:

The first option is to place, mix, and compact the aggregate materials for immediate use as a well graded stone base coarse. Once the foul weather clears, a sample of the material should be removed and the natural moisture content (NMC) determined. The ASTM microwave method will provide NMC information within 1 hour. The cement bags should then be placed and worked into the aggregate by a grader scarifier first to break-up the compacted maerial, and then by the soil stabilizer to ensure thorough mixing. The remainder of the procedure is identical to normal placement of material as discussed in sections 2.1.2 & 2.1.3.

The second option is the same as option #1 with exceptions as follow:

1. Once the onset of rain appears likely, a rain gauge (pluviometer) should be set up to record the amount of rainfall.

2. Using the amount of rainfall per square foot obtained from the rain gauge, the amount of water falling into the mixed material can be calculated.

3. The mixed material (fine and coarse aggregate)

should placed into the hole but not be compacted.

4. The cement bags should be spaced out, opened, and spread out.

5. The cement should be spread by hand and then mixed into the aggregate with the soil stabilizer.

6. Using the rainfall/SF value and the surface area of the layers (neglecting very minor additional area due to sloped sides the water added to the mix by rainfall should be calculated. The run-off of water from the surrounding pavement should be sealed off by placing small sand windrows on the upward slope slide of the layers.

7. The remainder of required water must be added by watertruck. During the field test conducted, it would have taken approximately one inch of rain to add all of the required amount of water to the layers. The dry material had a tremendous requirement for water approximately equal to 1.5 gal of water per cubic foot of material. The one inch of rain calculation is based on one lineal foot of the layers which was 5 ft wide and 8 inches thick which required 5 gallons of mixing water.

8. The layers must then be mixed by the soil stabilizer and compacted by the vibratory roller.

The time from the final measurement of the rain gauge to the compaction of the slab is critical. Williams reported that material that has been compacted and has a grade allowing some degree of run-off experiences a very low rate of absorption of water (9). Therefore, the effect of rain

can be minimized greatly by expedient construction once the cement is laid down.

If the cement powder has already been placed but water has not been added, then the procedure should be continued with the following exceptions:

1. Once the onset of rain appears likely, a rain gauge (pluviometer) should be set up to record the amount of rainfall.

2. Using the amount of rainfall per square foot obtained from the rain gauge, the amount of water falling into the mixed material should be calculated.

3. The remainder of required water must be added by watertruck.

4. Compaction must be expedited as much as possible.

If water and cement powder have been added before rainfall, then the construction must continue as normal except that compaction must be expedited as much as possible.

2.2.5 Calcium Chloride Application

The application of calcium chloride to accelerate curing/strength gain can be done by attaching a small container to the water truck that dispenses calcium chloride in solution into the water coming out of the watertruck. Appendix B contains more information on calcium chloride application. Calcium chloride was not used in the field test.

CHAPTER 3
MIX DESIGN DEVELOPMENT

3.1 Material Parameters

The method under study for performing rapid runway repair involves the use of roller compacted concrete, therefore, a roller compacted concrete mix design was needed.

3.1.1 Mix Design and Moist Rodded Unit Weight Input Values

The cement specific gravity and the aggregate types are given below:

1) Cement = Type III Rapid Hardening, SG = 3.15

2) Coarse Aggregate Type: Florida Limestone (#57 Stone)

3) Fine Aggregate Type: Polk Sand (Polk County Mine)

The parameters for the aggregates used in this project are listed below in Table 3.1.

Table 3.1 Aggregate Parameters

Aggregates	Fine	Coarse
Specific Gravity (SSD)	2.65	2.31
Absorption, %	0.70%	5.71%
Natural Moisture Content %	2.90%	2.70%
Dry Rodded Unit Wt., pcf	N/A	94.00
Moist Rodded Unit Wt., pcf	85.75	91.40
Absolute Unit Wt. (SSD) pcf	165.36	144.14
Fineness Modulus	2.20	N/A
Aggregate Size (inches)	N/A	0.75

The specific gravity (SSD) for the fine aggregate, absorption percentages for both aggregates, dry rodded unit weight for coarse aggregate, fineness modulus of fine aggregate, and aggregate size of coarse aggregate were obtained via Charles Allen of Florida Mining Company (10).

The specific gravity (SSD) for the coarse aggregate was obtained from Mr. Daniel Richardson, Civil Engineering Laboratory (11). Aggregate samples from material actually used in the test were obtained for the natural moisture content (12), moist rodded unit weight, and angle of repose tests conducted in-house at the Civil Engineering Laboratory.

3.1.1.1 Moist Rodded Unit Weight Test

The moist rodded unit weights were obtained using the test procedure as follows:

1) A 0.5 cf container which conforms to ASTM C29

specifications was weighed and recorded.

2) The fine aggregate was placed and rodded in the 0.5 cf container in accordance with ASTM C29.

3) The filled container was weighed and a unit weight (rodded unit weight) was obtained.

4) The filled container was then vibrated for 4 minutes while being topped off to keep the volume constant.

5) The filled container was weighed again and another unit weight (vibrated unit weight) was obtained.

6) Finally, the container was emptied and then filled again by pouring the fine aggregate in without any rodding or vibration. The filled container was weighed and a unit weight (unrodded unit weight) was obtained. The exact same procedure was repeated for the coarse aggregate.

3.1.1.2 Moist Rodded Unit Weight Calculations

Equation 3.1 was used to calculate the unit weight as shown below:

$$\text{Unit Weight} = \text{Sample Weight/Container Volume} \qquad (3.1)$$

$$= 41.75 \text{ lbs}/0.5 \text{ cf} = 83.5 \text{ pcf}$$

Table 3.2 lists the exact numbers obtained during these tests.

Table 3.2 Moist Rodded Unit Weight Test Results

Fine Aggregate	Unrodded	Rodded	Vibrated
Sample Weight (lbs)	41.75	44.00	51.10
Unit Weight (pcf)	83.50	88.00	102.20
Coarse Aggregate			
Sample Weight (lbs)	43.90	47.50	48.35
Unit Weight (pcf)	87.80	95.00	96.70

The final determination of the moist rodded unit weight for each aggregate was taken as the average of the unrodded and rodded unit weights for each aggregate. Therefore, the moist rodded unit weights for each aggregate were obtained in the same manner. Neglecting the vibrated unit weight and averaging the remaining rodded and unrodded unit weights was judged to be the value that would most closely simulate the dumping and raking action performed on the aggregate in the field test. The significant densification of the fine aggregate was judged as not accurate since the vibration required to cause densification would not occur in the field during actual aggregate placement. The above judgment on moist rodded unit weights is validated by calculation by showing that the void content's calculated from the moist rodded unit weights fall within acceptable ranges (13). This void content verification is shown in section 3.1.2.1 which follows.

The reason for the significant increase in moist rodded unit weight for the fine aggregate was sand bulking due to its moist state. The bulking was reduced during vibration which is why the fine aggregate experienced such a significant change (16% increase) in density through vibration. The larger particles (i.e. less surface area by weight than fine aggregate) of coarse aggregate are only minimally affected by bulking, therefore, no significant density occurs through vibration.

The equation used for the moist rodded unit weights obtained by averaging the rodded and unrodded unit weights is shown in equation 3.2.

MRUW = Moist Rodded Unit Weight

RUW = Rodded Unit Weight

UUW = Unrodded Unit Weight

$$MRUW = \frac{RUW + UUW}{2} \qquad (3.2)$$

For the fine aggregate the exact calculation utilizing equation 3.2 was as follows.

$$MRUW = \frac{88.00 \text{ pcf} + 83.50 \text{ pcf}}{2} = 85.75 \text{ pcf}$$

3.1.2 Analysis of Aggregate Parameters

The calculations in this section are performed for two purposes:

1) To verify by calculation the moist rodded unit weight numbers obtained through testing.

2) To show relationships between various aggregate parameters.

The terms listed below in Table 3.3 are defined for further calculations.

Table 3.3 Mix Design Variables

Vs	=	Volume of Solids
Vw	=	Volume of Water
Vv	=	Volume of Voids
Vt	=	Volume Total
e	=	Void Ratio = Vv/Vs
Ws	=	Weight of Solids
Ww	=	Weight of Water @ NMC
NMC	=	Natural Moisture Content = Ww/Ws
Wabw	=	Weight of Absorbed Water
Abs	=	Absorption = Wabw/Ws
Wa	=	Weight of Air = 0
SSDSG	=	Saturated Surface Dry Specific Gravity
FA	=	Fine Aggregate
CA	=	Coarse Aggregate
Gamma_W	=	Unit Weight of Water

The following calculations center around the phase diagram concept commonly found in geotechnical engineering. Its adaptation to use with aggregate parameters provided the required mathematical and conceptual model to check the accuracy of aggregate parameters. The schematic of a phase diagram is shown below in figure 3.1.

Vt		Va = Value	Air	Wa = 0	
	Vv	Vw = Value	Water	Ww = Value	Wt
		Vs = Value	Solid	Ws = Value	

Figure 3.1 Phase Diagram Relationships (14)

3.1.2.1 Verifying Moist Rodded Unit Weight

The calculations to verify the moist rodded unit weights began by calculating the weight of solids (Ws), volume of solids (Vs) and finally the void content that must exist in the moist rodded unit weight value. The void content that is calculated is then checked to ensure it falls within established standards (15).

$$MRUW = \frac{Ws + Ww}{Vt} \qquad (3.3)$$

$$Ww = NMC(Ws) \qquad (3.4)$$

$$MRUW = \frac{Ws + NMC(Ws)}{Vt} \qquad (3.5)$$

$$MRUW = \frac{Ws(1 + NMC)}{Vt} \qquad (3.6)$$

Rearranging equation 3.6 yielded equation 3.7.

$$Ws = \frac{Vt(MRUW)}{(1 + NMC)} \qquad (3.7)$$

Utilizing equation 3.7 with an assumed total volume (Vt) of 1 cf, the fine aggregate calculation was as follows.

Assumed: Vt = 1.00 cf

Given: NMCFA = 2.90% (16)

MRUW = 85.75 pcf

$$WsFA = \frac{1.00\ cf(85.75\ pcf)}{(1 + .029)}$$

WsFA = 83.33 lbs

Similarly the weight of solids for the coarse aggregate (WsCA) equaled 89.00 lbs. Continuing on with the fine aggregate brings on the volume of solid calculation (VsFa). Equation 3.8 details the saturated surface dry specific gravity relationship to the volume of solids (Vs).

$$SSDSG = \frac{Wabw + Ws}{Vs * Gamma\_W} = \frac{Ws(1 + Abs)}{Vs * Gamma\_W} \qquad (3.8)$$

Rearranging equation 3.8 yields equation 3.9 which was used to calculate the volume of solids.

$$Vs = \frac{Ws(1 + Abs)}{SSDSG * Gamma\_W} \qquad (3.9)$$

$$VsFA = \frac{83.33\ lbs(1 + 0.007)}{2.65(62.4\ pcf)}$$

VsFA = 0.51 cf

Similarly, the volume of solids for the coarse aggregate (VsCA) equals 0.65 cf.

Calculating the volume of the voids for both aggregates is done by equation 3.10.

$$Vv = Vt - Vs \qquad (3.10)$$

Therefore the volume of voids calculation for the fine aggregate was as follows.

$VvFA = 1 - 0.51 = 0.49$ cf

Similarly, the volume of voids for the coarse aggregate equaled 0.35 cf. The void content values (VvFA & VvCA) fell within expected ranges of 30 - 45% for coarse aggregates and 40 - 50% for fine aggregates (17).

<u>3.1.2.2 Moist Rodded Unit Weight Ranges</u>

Working backwards through equations (3.10, 3.9 and 3.7) at the void ratio ranges of 30 - 35% for the coarse aggregates and 40 - 50% for the fine aggregates, yields moist rodded unit weight ranges of 98.0 - 77.0 pcf for the coarse aggregate and 101.4 - 84.5 pcf for the fine aggregate.

<u>3.1.2.3 Void Ratios</u>

The void ratios (e) were calculated using equation (3.11).

$$e = \frac{Vv}{Vs} \qquad (3.11)$$

The values calculated for the fine and coarse aggregates are listed below.

Void Ratio (FA) = 0.97

Void Ratio (CA) = 0.53

3.1.2.4 Aggregate Parameter Ranges

The ranges obtained from the Florida Mining Company (18) for absorption and the fineness modulus are listed below in Table 3.4.

Table 3.4 Aggregate Parameter Ranges

Aggregates	Fine		Coarse	
	High	Low	High	Low
Absorption, %	0.71%	0.69%	5.82%	5.60%
Fineness Modulus	2.4	2.0		

3.2 Proportioning of Mix Design

3.2.1 Calculating Proportions by Absolute Volume in Mix

The mix design follows the Maximum Density Approach set forth in the ACI Code (19).

Step (1)

Set air-free volume of paste to air-free volume of mortar (Pv) equal to suitable value for interior mass mix in accordance with ACI Code (20).

$$Pv = 38.0\%$$

Step (2)

Select flyash/cement (F/C) and water/(cement + flyash) (W/(C+F)) VOLUME ratios from ACI Code (21).

$$F/C = 0.0\%$$

$$\text{Volume } W/(C+F) = 154.0\%$$

Step (3)

Select volume of coarse aggregate (Vca) by selection

from ACI Code (22) by equating #57 approximately equal to 3/4" maximum size aggregate (23).

$$Vca = 49.0\%$$

Step (4)

Calculate the volume of air-free mortar per cubic yard (Vm) assuming 2% entrapped air (Va) (24) using equation (3.12).

$$Va = 2.0\%$$

$$Vm = Cv * (1 - Va) - Vca \qquad (3.12)$$

$$Vm = 49.0\%$$

where:

Cv = the unit volume of concrete = 1 cy
= 27 cf

Step (5)

Calculate the air-free paste volume (Vp), using the selected paste volume ratio (Pv) of Step (1) and equation (3.13).

$$Vp = Vm * Pv \qquad (3.13)$$

$$Vp = 18.6\%$$

Step (6)

Determine the fine aggregate volume (Vfa) using equation (3.14).

$$Vfa = Vm * (1 - Pv) \qquad (3.14)$$

$$Vfa = 30.4\%$$

Step (7)

Determine the trial water volume (Vw) with equation 3.15.

$$Vw = Vp * [(W/(C+F))/(1+W(C+F))] \qquad (3.15)$$

Vw = 11.3%

Step (8)

Determine cement volume (Vc) with equation 3.16.

$$Vc = Vw/[(W/(C+F))(1+F/C)] \quad (3.16)$$

$$Vc = 7.3\%$$

3.2.2 Converting Absolute Volumes to Saturated Surface Dry Weights

The weight of aggregate required in a saturated surface dry condition (WSSDFA) is calculated by equation 3.17, 3.18.

$$WSSD = Ws(1 + Abs) = Vs * SSDSG * Gamma\_W \quad (3.17)$$

Vs = Vfa or Vca (% Volume of aggregate/cy of mix)

$$WSSD = Vs/100 * 27 \text{ cf/cy} * SSDSG * Gamma\_W \quad (3.18)$$

$$WSSDFA = 30.4/00 * 27 \text{ cf/cy} * 2.65*62.4\text{pcf}$$

$$WSSDFA = 1356 \text{ lbs}$$

Similarly, the weight of coarse aggregate in a saturated surface dry condition (WSSDCA) equals 1907 lbs.

The weight of mixing water required (Wmwr) for aggregates at a SSD condition is calculated by equation 3.19.

$$\text{Volume of Water} = Vw/100 = 11.3\%/100$$

$$= 0.113 \text{ cy}$$

$$Wmwr = Vw/100 * Gamma\_W \quad (3.19)$$

$$= 0.113 \text{ cy} * 27 \text{ cf/cy} * 62.4 \text{ pcf}$$

Wmwr = 190.2 lbs

The weight of cement (Wc) is calculated by equation 3.20.

SGC = Specific gravity of portland cement type III.

$$Wc = Vc/100 * 27\ cf/cy * SGC * Gamma\_W \tag{3.20}$$

$$= 7.3/100 * 27 * 3.15 * 62.4$$

Wc = 389 lbs

3.2.3 Converting Saturated Surface Dry Weights to Natural Moisture Content Weights

The first step is to calculate the water added to the mix by the aggregates. This requires calculating the dry weight of the aggregate (Ws). Ws is calculated by rearranging equation 3.9 into equation 3.21.

$$Vs = \frac{Ws(1 + Abs)}{SSDSG * Gamma\_W} \tag{3.9}$$

$$Ws = \frac{Vs * 27cf/cy * SSDSG * Gamma\_W}{(1 + Abs)} \tag{3.21}$$

Vs = Vfa or Vca

$$WsFA = \frac{0.304 * 27\ cf/cy * 2.65 * 62.4\ pcf}{(1 + 0.007)}$$

WsFA = 1347 lbs

Similarly, the dry weight of coarse aggregate (WsCA) equals 1804 lbs.

The weight of mixing water contributed by aggregates (Wmwca) is calculated by equation 3.22.

$$Wmwca = Ws_i * (NMC_i - Abs_i) \qquad (3.22)$$

Coarse:

$$1804 \text{ lbs} * (0.027 - 0.057) = -54.3 \text{ lbs}$$

Fine Agg:

$$1347 \text{ lbs} * (0.029 - 0.007) = 29.6 \text{ lbs}$$

$$Wmwca = -24.7 \text{ lbs}$$

The weight of adjusted mixing water required (Wamwr) is calculated by equation 3.23.

$$Wamwr = Wmwr - Wmwca \qquad (3.23)$$

$$= 190.2 \text{ lbs} - (-24.7 \text{ lbs})$$

$$= 214.9 \text{ lbs}$$

The weight of aggregates at natural moisture content (WNMC) is calculated by equation 3.24.

$$Ws * (1 + NMC) = WNMC \qquad (3.24)$$

Coarse Agg (WNMCCA)

$$1804 \text{ lbs} * 1.027 = 1853 \text{ lbs}$$

Fine Agg (WNMCFA)

$$1347 \text{ lbs} * 1.029 = 1386 \text{ lbs}$$

The weight of portland cement emains the same when going from SSD conditions to NMC conditions. Table 3.5 lists weights and weight percentages of ingredients after moisture adjustment at NMC. These are the amounts actually put in place per cubic yard of mix. Moisture adjustment means the

amounts are adjusted for moisture pre-exsting or lacking in the aggregates. The relationship between SSD weights and NMC weights can be seen as the numbers are listed side by side.

Table 3.5 Actual Mix Design Summary (1 Cy)

	Theoretical Weight @ SSD	Actual Weight @ NMC	Weight% @ NMC
Slump (inches)	0.0 in	0.0 in	N/A
Air Content %	2%	2%	N/A
Water	190.2 lb	214.9 lb	5.6%
Cement	389.0 lb	389.0 lb	10.1%
Coarse Aggregate	1907 lb	1853 lb	48.2%
Fine Aggregate	1356 lb	1386 lb	36.1%
Totals	3843 lb	3843 lb	100.0%
Unit Weight	142.3 pcf	142.3 pcf	N/A

Table 3.6 lists the absolute volumes, SSD weights (WSSD), Dry weights (Ws), weight percentages, and dry volumes (Vs). Table 3.6 is provided for convenient referencing.

Table 3.6 Volumes and Weights of Ingredients.

	Absolute Volumes	WSSD Weights	Ws (DRY) Weights	DRY Weight %	Vs Vol(cF)
Water -	11.3%	190.2	302.6	7.9%	3.0
Cement -	7.3%	389.0	389.0	10.1%	2.0
Coarse -	49.0%	1907.0	1804.0	46.9%	13.2
Air -	2.0%	0.0	0.0	0.0%	0.5
Fine -	30.4%	1356.4	1347.0	35.1%	8.2
Sums -	100.0%	3842.7	3842.7	142.3	27.0

Sums equal, (water shifts)

3.2.4 Calculating the Water/Cement (W/C) Ratio and Moisture Content of Mix

The water to cement ratio (W/C) is calculated by equation 3.25.

$$W/C = WSSD/Wc \qquad (3.25)$$

$$W/C = 190.2 \text{ lbs}/389.0 \text{ lbs}$$

$$W/C = 48.89\%$$

The overall moisture content of the mix should be approximately 5% (+/- .3%) for the mix to ensure adequate stiffness. That translates to 180 to 200 lbs of mixing water for a mix design with unit weight from 140 - 145 pcf (25).

The moisture content of the mix (MCM) is calculated by equation (3.22).

$$Gamma\_M = \text{Unit weight of mix} = 142.3 \text{ pcf}$$

$$MCM = WSSD/(27 * Gamma\_M) \qquad (3.22)$$

$$MCM = 190.2 \text{ lbs}/(27 * 142.3)$$

$$MCM = 4.95\%$$

CHAPTER 4
LAYER THICKNESS CALCULATION

4.1 Introduction

This chapters illustrates the procedure for converting weights to pre-mix volumes which takes into account the bulking of materials that are in a partially saturated condition. This is required since material in the field will very rarely be oven-dry nor 100% saturated. The determination of the general shape of the crater to be repaired is discussed followed by the conversion from pre-mix volumes to layer thicknesses.

It is very important at this point to undestand how the materials will be handled to create the concrete mix that forms the slab. The coarse aggregate required will first be placed upon the ground as a homogeneous layer. The cement powder bags will then be placed, opened, and emptied. The cement will then be spread forming a thin layer on top of the coarse aggregate. The sand will then be placed on top of the cement layer forming a homogeneous sand layer. The layers will then be mixed using a large rotor tiller such as is commonly used in soil stabilization. For more detailed construction information, refer back to Chapter two of this report.

The discussion of the type of crater includes the discussion of the equivalent hole concept (26). The equivalent hole concept involves all of slab material (aggregates and cement) required for the crater to be placed in layers that do not completely fill the crater or are

outside of the crater. The dimensions of the layers can vary from the dimensions of the actual crater, as long as the total volume of material in the layers (i.e. equivalent hole) equals the volume of material required to make the slab in the crater. The procedure is detailed below to provide more clarity about this concept.

The equivalent hole concept begins by determining the volume of actual crater to be filled. The depth of the crater to be filled with concrete was set to 10 inches. Therefore, only the top surface area of the crater will affect the volume calculation. Once the volume of material for the actual crater is calculated, it is used to calculate dimensions of the equivalent hole. Two dimensions are arbitrarily set, the third dimension is then calculated so that the equivalent hole volume equals the actual crater volume. For example, if the equivalent hole depth is arbitrarily set at 6 ins and the equivalent hole width is arbitrarily set at 5 ft., then the equivalent hole length is calculated to allow the equivalent hole volume to equal the actual crater volume. The material will have to be moved into place in the actual crater by motor graders and/or front end loaders.

4.2 Converting NMC Weight Proportions to Pre-mix Volumes

The aggregates will be proportioned in the field based on pre-mix volumes. The moist rodded unit weights (MRUW) were obtained using the aggregates at their NMC. Having the

mix proportions by weight at NMC, allows the conversion to pre-mix volumes (PMV) using the MRUW. The conversion is shown by equation 4.1.

$$PMV = WNMC * 1/MRUW * 0.037\ cy/cf \quad (4.1)$$

The calculation for the coarse aggregate is shown below.

$$PMVCA = 1853\ lbs * 0.01094\ cf/lbs * 0.037\ cy/cf$$
$$= 0.751\ cy$$
$$= 0.751\ cy/cy\ mix = 20.3\ cf/cy\ mix$$

Similarly, the pre-mix volume of fine aggregate (PMVFA) equals 0.599 cy/cy mix.

The pre-mix volume of water (PMVW) is shown by equation 4.2.

Wamwr = weight of adjusted mixing water required

$$PMVW = Wamwr/Gamma\_W \quad (4.2)$$
$$= 214.9\ lbs/62.4\ pcf$$
$$= 0.128\ cy/cy\ mix$$

The pre-mix volume of cement (PMVC) is calculated by equation 4.3.

$$PMVC = Vc/100 * 1.3 \quad (4.3)$$
$$= 7.3/100 * 1.3$$
$$= 0.095\ cy/cy\ mix$$

The 1.3 factor takes into account the fluffing up of the powder due to opening bags and placing (i.e. voids between cement particles). The amount of cement placed is measured by weight converted into a number of 94 lb bags.

The pre-mix volume of cement is only required to allow an accurate measure of the combined height of the fine, coarse, and cement layers.

4.3 Converting Pre-mix Volumes to Layer Thicknesses

4.3.1 Calculating Crater Type and Size

The first requirement is to identify the general shape of the crater. Most craters will fall into a circular or rectangular shape. A section is devoted to irregular crater shapes. The slab depth is independent of the crater size. The slab depth may be set to the existing slab depth plus one or two inches to provide a safety margin against running short of material.

4.3.1.1 Circular Crater Parameters

For a circular crater with a surrounding slab depth of 8 inches, the replacement slab is set at 10 inches as detailed in the example set of parameters below in Table 4.1 for a circular crater.

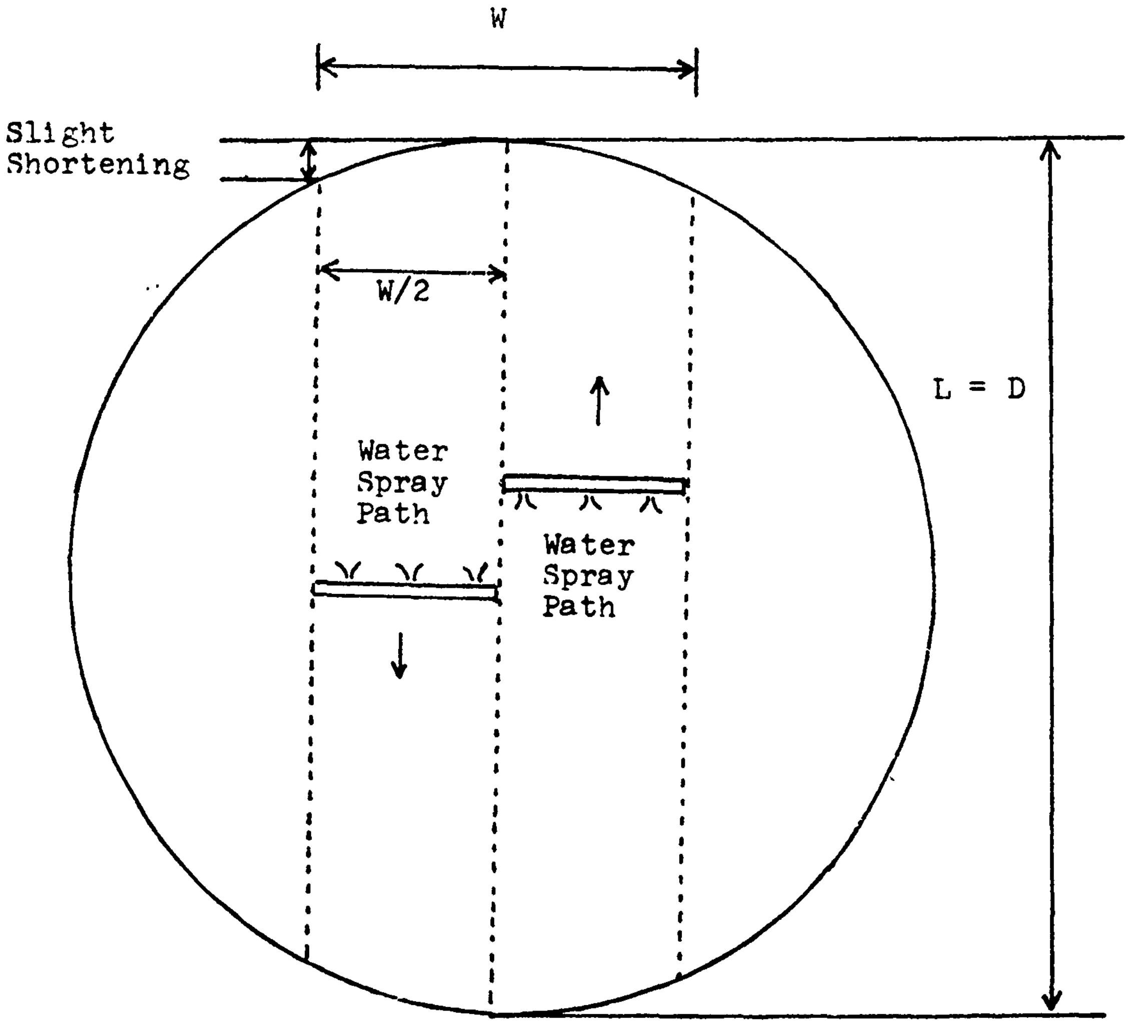
Table 4.1 Circular Crater Parameters

Slab Depth	= 10 in
Diameter	= 40 ft
Area	= 1257 SF
Slab Volume	= 38.8 Cy

The slab volume can be converted to an equivalent hole that can be placed within the crater. See Figure (4.1) for an illustration. The water application will be discussed in detail in a later chapter, however, for

now it is essential to know whether the water spray bar width can be adjusted or not. If the water spray bar width cannot be adjusted then the equivalent width of the crater should be set as some multiple of the watertruck spray bar width. This will prevent overlapping water during the water application phase. Therefore, assuming a spray bar width of 10 ft, the slab width could be set at 20 ft. If the spray bar width is adjustable, then the equivalent width should be set as some multiple of the optimum width for mixing by the mixer. The optimum width for the mixer is the largest width the mixer can thoroughly mix on one pass.

Since the crater is circular, the equivalent length can be set as the diameter realizing that the length will slightly shorter on sides away from the center (see Figure 4.1). The minimum of one inch of extra slab thickness provides for at least 10% extra material (1 inch/9 inches = 11%, 1 inch/8 inches = 12.5%) which will ensure enough material is placed to compensate for this slight shortening. Obviously, the extra two inches selected will provide for even more excess material. The slight shortening will typically cause less than a 5% decrease in volume. See Figure 4.2 for more detail on the volume reduction due to shortening.



The Equivalent Width (W) is equal to two Spray Bar Widths

The Equivalent Length (L) is the Diameter

Figure 4.1 Circular Crater Equivalent Hole

The equivalent width and depth are shown in Table 4.2 below.

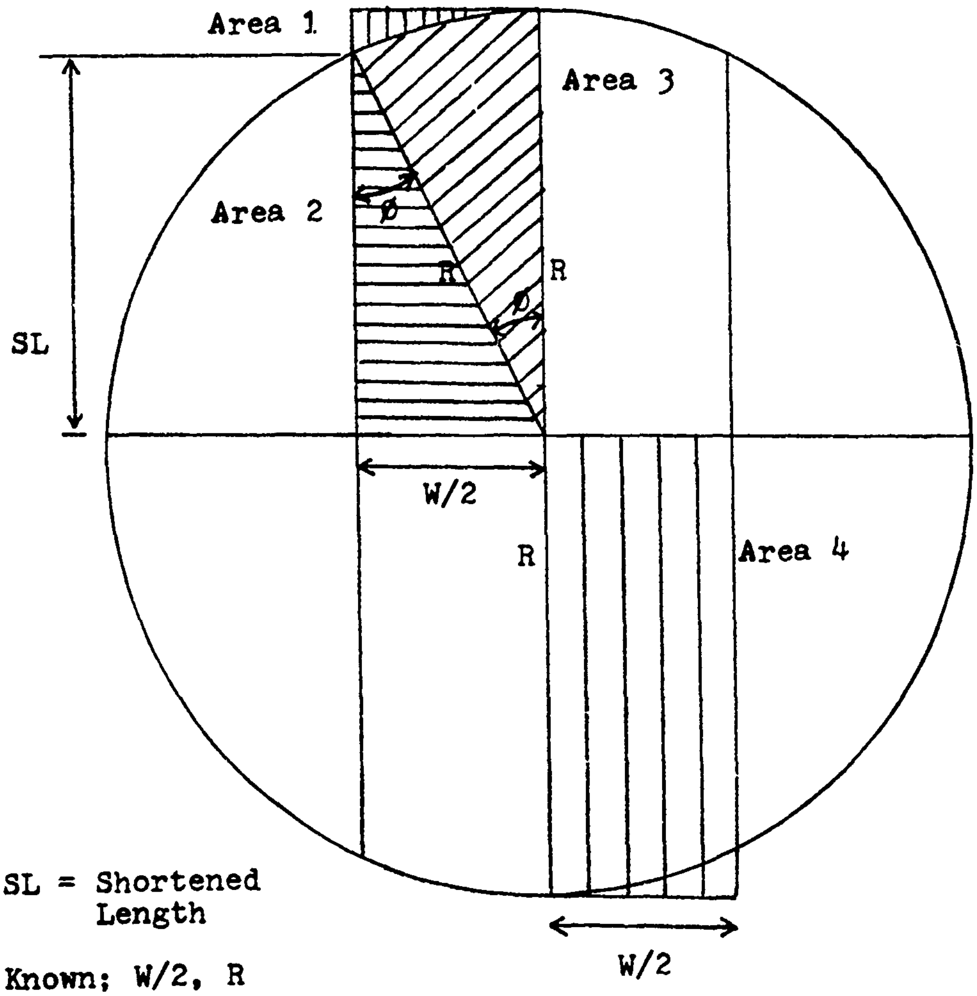
Table 4.2 Volume Reduction for Equivalent Hole

Equiv. Width = 20 ft
Equiv. Len. = 40 ft

The equivalent depth is calculated by equation 4.4.

$$\text{Equiv. Depth (ft)} = \frac{\text{Slab Volume (cy)} * 27 \text{ cf/cy}}{\text{Equiv. Wid.(ft)} * \text{Equiv. Len.(ft)}} \tag{4.4}$$

The equivalent depth will be greater than the actual slab depth. If the equivalent slab depth is thicker than can mixed by the mixer, than the equivalent length should be increased until the equivalent depth reaches a thickness that can be mixed. Use equation 4.4 to calculate the new equivalent depth.



SL = Shortened Length

Known; W/2, R

Find; Area 1

Solution:

Calculate ϕ;	$\sin \phi = (W/2)/R$	Solve for ϕ
Calculate SL;	$\cos \phi = SL/R$	Solve for SL
Calculate Area 4;	Area 4 = (W/2) * R	Solve for Area 4
Calculate Area 3;	Area 3 = pi * R^2 * (ϕ/360)	Solve for Area 3
Calculate Area 2;	Area 2 = 1/2 * SL * W/2	Solve for Area 2
Calculate Area 1;	Area 4 = Area 1 + Area 2 + Area 3	Solve For Area 1

Note; On a perfect circle, Area 1/Area 4 less than 5% normally

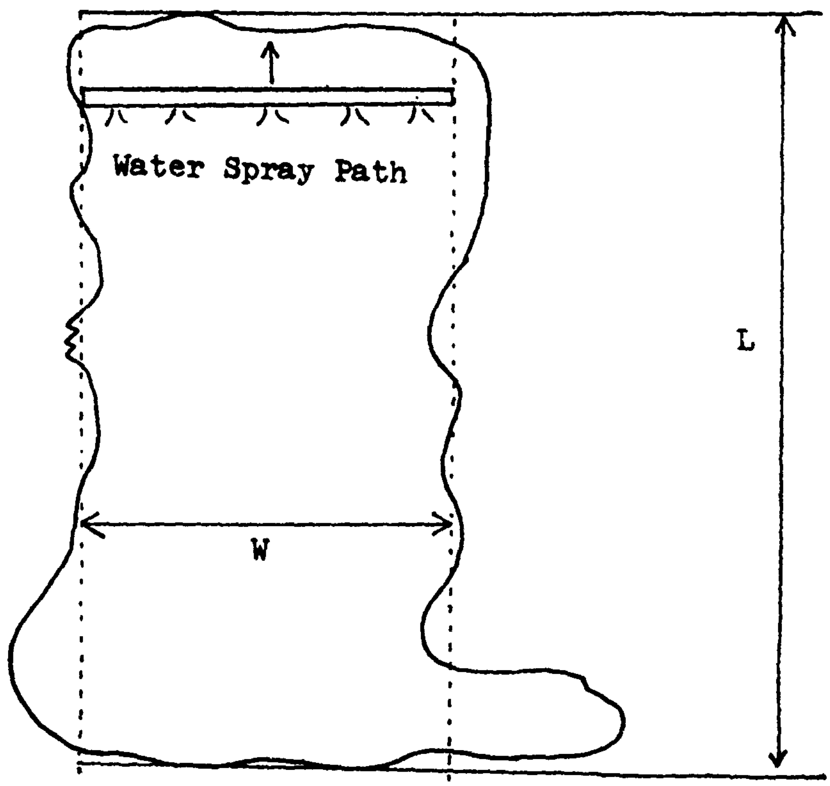
Figure 4.2 Volume Reduction For Equivalent Hole

4.3.1.2 Rectangular Crater Parameters

A crater may come closer to resembling a rectangular crater rather than a circular one. In that event, the actual crater volume must be calculated by breaking the crater area down into smaller geometric shapes, calculating the areas of the individual geometric shapes and summing the areas to obtain the total crater area. The crater area can then be multiplied by

The 10 inch slab thickness to obtain the crater volume.

The crater volume can then be used to obtain equivalent parameters. The use of an equivalent hole can be accomplished by setting the equivalent width equal to a multiple of the spray bar width. The equivalent length can be set as the longest straight run through the hole. See Figure (4.3) for more detail on the equivalent parameters. The equivalent depth can then be calculated using equation 4.4 above.



W = Equivalent Width

L = Equivalent Length

Figure 4.3 Rectangular Crater Equivalent Hole

4.3.1.3 Irregular Crater Parameters

If the crater is very irregular in shape, the crater volume can still be calculated by breaking down the crater into smaller geometric shapes. The equivalent hole concept can then be used to determine the dimensions of an equivalent hole. If placement of the layers within the hole is difficult, then the material may be placed in layers in an equivalent hole outside of the actual crater.

4.3.1.4 External Equivalent Hole Parameters

The field test conducted to test this rapid runway repair method made use of the external equivalent hole concept discussed in this section. The use of an equivalent hole completely outside of the crater has benefits and disadvantages over placing the material directly in the hole.

A key difference between this method and regular soil stabilization is that in normal soil stabilization operations the mixer can only reach down to a maximum depth of between 6 - 12 inches before mixing becomes poor. The equivalent hole concept involves all of the material being laid above ground on existing pavement. Thick layers would be screeded off by the tractor underbelly as the tractor towed the mixer behind it. The screeding off should be limited to 2 - 3 inches or else the tractor may experience difficulty traveling over the material. The potential difficulties being

material falling into the engine compartment through the front grill of the tractor and uplift by material being forced underneath the tractor causing the tractor to lose traction. Therefore, the combined layer thicknesses before mixing should only be 2 - 3 inches above minimum ground clearance of the tractor towing the mixer. Th minimum ground clearance is usually approximately 12 inches (27). By placing the material outside of the hole and onto the existing pavement, the use of a level bottom surface canbe taken advantage of to measure the layer heights.

The principal disadvantage is that the material must be moved farther to get it into the crater. Further discussion on advantages and disadvantages of the equivalent hole will be discussed in later chapters.

The sequence of steps is the same as for determining an equivalent hole within a crater. The actual crater volume is deterined based on the actual crater's dimensions.

The field test conducted to test this runway repair method used this external equivalent hole concept. The actual test crater was a 9 ft * 30 ft * 8 inch rectangular crater. The depth was increased by 2 inches to ensure enough material and provide for some extra material to fil small extraneous holes nearby the test hole.

Therefore the actual crater dimensions for the test were set at 9 ft * 30 ft * 10 inches yielding an actual crater volume of 8.33 cy.

The water spreading width could easily be adjusted to any width thereby eliminating the need to set the equivalent width as a multiple of the water spray bar width. Therefore, the equivalent width was set based on the maximum width the mixer could thoroughly mix. The maximum width was determined to be 5 ft to ensure excellent mixing of the aggregate and cement layers. The depth was arbitrarily set at 6 inches. Therefore, by switching sides of the equivalent depth and length terms in equation (4.4), the equivalent length was calculated to be 90 ft. Table 4.3 below summarizes the actual and equivalent parameters of the test hole.

Table 4.3 Actual and Equivalent Parameters of TestHole

	Actual	Equivalent
Depth	10 in	6 in
Width	9 ft	5 ft
Length	30 ft	90 ft
Volume	8.3 cy	8.3 cy

Once the actual volume of the crater was determined, the total quantities of material required could be calculated by multiplying the values required

for 1 cy of mix by the total volume of 8.3 cy. Therefore the values of Table 3.6 are reproduced and increased by 8.3 to show a summary of total material required by weight to fill the testhole.

Table 4.4 Actual Mix Design Summary (1 cy & 8.3 Cy)

	Actual Weight @ NMC (1 cy)	Actual Weight @ NMC (8.3 cy)
Slump (inches)	0.0 in	
Air Content %	2%	
Water	214.9 lb	1791 lb
Cement	389.0 lb	3242 lb
Coarse Aggregate	1853 lb	15,442 lb
Fine Aggregate	1386 lb	11,550 lb
Totals	3843 lb	32,025 lb
Unit Weight	142.3 pcf	142.3 pcf

4.3.2 Calculating Pre-Mix Layer Thicknesses

Once the dimensions of the crater to be filled were determined, the layer thicknesses were calculated. The layer thicknesses were based on the equivalent dimensions. If equivalent dimensions had not been calculated (i.e. no equivalent hole was used), then the actual dimensions would have been used.

The steps for calculating layer thickness were as follows:

1. The volume of 1 cy (27 cf) was divided by the

equivalent depth, thereby determining the surface area covered by 1 cy of mix. Equation 4.5 below was used to perform the calculation.

Area = 27 cf/Equivalent Depth * 12 in/ft (4.5)

Using equation 4.5 the area was calculated.

Area = 27 cf/6 inches * 12 inches/ft

Area = 54 SF

2. The pre-mix volume (PMV) was converted from cy to cf, divided by the surface area and multiplied by 12 to obtain the pre-mix layer thickness (PMLT) in inches. Equation 4.6 below was used to perform the calculation.

PMLT = PMV * 27 cf/cy * 1/Area * 12 in/ft (4.6)

Using equation 4.6, the pre-mix layer thickness for the coarse aggregate (PMLTCA) was as follows.

PMLTCA = 0.751 cy * 27 cf/cy * 1/54 SF * 12 in/ft

PMLTCA = 4.5 ins

Similarly the fine aggregate and cement powder thicknesses were 3.6 ins and 0.6 ins respectively.

4.3.3 Angle of Repose

As discussed the material was laid down in homogeneous layers and then blended together by a soil stabilizer. Since there was no containment along the sides, each layer formed a trapezoid. The grades of the sides of the trapezoidal layers are commonly referred to as angles of repose in geotechnical engineering (28).

The angles of repose for the coarse aggregate and fine aggregate were measured based on techniques shown in Holtz

(29). Table 4.5 contains the angles of repose for the aggregates.

Table 4.5 Angles of Repose

Material	Angle
Coarse Aggregate	30 degrees
Fine Aggregate	39 degrees *

* High angle value due to bulking of moist sand.

4.3.4 Calculating Angle of Repose Layer Thicknesses

When the equivalent dimensions were calculated from the crater volume, the assumption was that the layers would form a rectangular box. The key concept is that the bottom of the bottom layer width equaled the equivalent width (i.e. at 0 inches height, the width equaled 5 ft), however, the layer width decreased (i.e. width < 5 ft) as the layer height increased (i.e. height > 0 inches, moving up the trapezoid). Therefore, in order to maintain the same volume one of the other dimensions must be increased. The length was arbitrarily selected to remain constant, therefore, the layers depths were increased.

Since the length was 90 ft and the width was only 5 ft, the depth increase due to the trapezoidal sides was considered significant. The depth increase due to the trapezoidal ends was considered insignificant due to the short end lengths which were over an order of magnitude ss than side lengths; 5 ft < 50 ft < 90ft.

The angle of repose layer thicknesses were calculated in the order of coarse aggregate on bottom, cement, and then fine aggregate on top. Figure (4.4) shows the order of placement of aggregates.

```
          ________________
         / Fine Aggregate \
        /__________________\
       /      Cement        \
      /______________________\
     /    Coarse Aggregate    \
    /__________________________\
```

Figure (4.4)

The following terms are defined in Table 4.6 for use with angle of repose layer thickness equations.

Table 4.6 Angle of Repose Variable Definitions

CA	=	Coarse Aggregate
FA	=	Fine Aggregate
ARLT	=	Angle of Repose Layer Thickness
PMLT	=	Pre-Mix Layer Thickness
CraWidth	=	Equivalent Width
	=	Width of Bottom Layer
RedCraWid	=	Reduced Top of Coarse Layer Width

The basic relationship is that the assumed rectangular volume must equal the actual trapezoidal volume as detailed in equation 4.7.

$$\frac{\text{CraWidth} * \text{PMLTCA}}{12} = \frac{\text{CraWidth} + \text{RedCraWid}}{2} * \frac{\text{ARLTCA}}{12} \tag{4.7}$$

The following equations are provided to illustrate the derivation to obtain the final quadratic equation for angle of repose layer thicknesses (ARLTs).

$$\text{RedCraWid} = \text{CraWidth} - 2 * \frac{\text{ARLTCA}}{\text{Tan(ARCA)} * 12} \qquad (4.8)$$

Substituting equation 4.8 into the right half only of equation 4.7 for RedCraWid is detailed below.

$$\frac{2 * \text{Crawidth} - 2 * \text{ARLTCA}/(\text{Tan(ARCA)} * 12)}{2} * \frac{\text{ARLTCA}}{12}$$

The above right half of equation 4.7 simplifies as shown below.

$$\left[\text{CraWidth} - \frac{\text{ARLTCA}}{12\ \text{Tan(ARCA)}}\right] * \frac{\text{ARLTCA}}{12}$$

Combining the above simplified right half of equation 4.7 back with the left half yields the following.

$$\frac{\text{CraWidth} * \text{PMLTCA}}{12} = \frac{\text{CraWidth} * \text{ARLTCA}}{12} - \frac{(\text{ARLTCA})^2}{\text{Tan(ARCA)}*144}$$

Further simplification yields equation 4.9 as a quadratic expression detailed below.

$$\frac{-(\text{ARLTCA})^2}{\text{Tan(ARCA)} * 12} + \text{ARLTCA(CraWidth)} - \text{PMLTCA(Crawidth)} \qquad (4.9)$$

For the coarse aggregate the a, b, c terms are detailed below in Table 4.7.

Table 4.7 Coarse Aggregate Quadratic Terms

a = -1/(tan(ARCA)*12)	
b = Crawidth (ft)	
c = -PMLTCA(Crawidth)	PMLTCA in inches

The values calculated for a, b, c and the roots are detailed below in Table 4.8.

Table 4.8 Coarse Aggregate Quadratic Terms and Root Values

a	=	-0.14434
b	=	5.00000
c	=	-22.50000
Root#1	=	5.32 in
Root#2	=	29.33 in

The correct root is root #1 with the angle of repose layer thickness for coarse aggregate (ARLTCA) equal to 5.32 inches which presents a slight increase from the pre-mix layer thickness for coarse aggregate (PMLTCA) of 4.5 inches. Root #2 is obviously far above 4.5 inches and therefore could not possibly be correct. The quadratic will routinely work out with one reasonable and one unreasonable value.

The fine aggregate general equation is detailed below in equation 4.10.

$$\frac{CraWidth + PMLTFA}{12} = \frac{TopFAWid + RedCraWid}{2} * \frac{ARLTFA}{12} \qquad (4.10)$$

Where; RedCraWid = Reduced Top of Coarse Layer Width (Same as above in coarse)

TopFAWid = Reduced Top of Fine Layer Width

The RedCraWid value remains the same as detailed using equation 4.8. The top of the fine aggregate layer width (TopFAWid) was calculated as detailed below in equation 4.11.

$$TopFAWid = RedCraWid - \frac{2 * (ARLTFA)}{Tan(ARFA) * 12} \qquad (4.11)$$

Substituting equation 4.11 into equation 4.10 yields equation 4.12 which is a quadratic expression for calculating the angle of repose layer thickness for the fine aggregate (ARLTFA). Two intermediate algebraic steps have been omitted for brevity.

$$\frac{-(ARLTFA)^2}{Tan(ARFA) * 12} + RedCraWid(ARLTFA) - PMLTFA(Crawidth) \qquad (4.12)$$

For the fine aggregate the a, b, c terms are detailed below in Table 4.9.

Table 4.9 Fine Aggregate Quadratic Terms

a = -1/(tan(ARFA)*12)		
b = RedCraWid = Top of coarse layer width (ft)		
c = -PMLTFA(Crawidth)	PMLTFA in inches	

The values calculated for a, b, c and the roots are detailed below in Table 4.10.

Table 4.10 Fine Aggregate Quadratic Terms and Root Values

a =	-0.10291
b =	3.46829
c =	-17.93980
Root #1 =	6.41 in
Root #2 =	27.29 in

The correct root is root #1 with the angle of repose layer thickness for the fine aggregate (ARLTFA) equal to 6.41 inches which presents an increase from the pre-mix layer thickness for the fine aggregate (PMLTFA) of 3.6 inches. The narrowing of the base layer of coarse aggregate underneath the fine aggregate magnifies the angle of repose effect.

The angle of repose effect is neglected for the cement layer due to the thinness of the cement layer at 0.6 inches. Table 4.11 contains a listing of pre-mix volumes (PMVs), pre-mix layer thicknesses and angle of repose layer thicknesses:

Table 4.11 Layer Thicknesses

	PMV (cy)	PMLT (in)	ARLT (in)
Coarse Aggregate *	0.751	4.5	5.3
Cement	0.095	0.6	0.6
Fine Aggregate *	0.599	3.6	6.4
Water	0.128	0.8	0.8
Total Pre-Mix Thickness =		8.7	12.3

Water layer does not add to pre-mix thickness.

4.4 Calculating Spacing of Cement Bags

The following data is provided in Table 4.12 for calculations in this section.

Table 4.12 Cement Powder Data

One bag cement	= 94 lbs (lbs cement/bag)
Cement/Cy mix	= 389 lbs (lbs Cement/Cy mix)
Cy mix/one bag	= 0.242 cy = 6.524 cf
Equiv. Depth	= 0.500 ft

Type III high early strength cement was used which comes in 94 lb bags. The amount of cement per cy of mix is calculated in section 3.2. The cubic yards of mix (Cy mix) per bag of cement was calculated by dividing the weight of one bag of cement (94 lbs) by the total weight of cement in one cubic yard of concrete (389 lbs). The equivalent depth was determined to be 6 inches or 0.5 ft in section 4.3.1.4 as shown in Table 4.3 of section 4.3.1.4.

The area that 1 cy of concrete will cover at a depth of 6 inches equals 54 SF. The amount of square footage that one bag of cement provides is detailed by equations 4.13 & 4.14.

Cy Area = 1 cy * 27cf/cy/0.5 ft (4.13)

Cy Area = 54 SF

1 Bag Area = 54 SF/cy * 0.242 cy mix/bag (4.14)

1 Bag Area = 13.1 SF

The equivalent width was calculated at 5 ft, therefore the bag spacing was calculated by equation 4.15.

Bag Spacing = 1 Bag Area/Equiv Width (4.15)

Bag Spacing = 2.62 ft

The bag spacing was reduced to 2.5 ft for convenience of placement in the field. Table 4.13 summarizes the above calculations.

Table 4.13 Cement Bag Spacing

	Ratio	Area SF	L (ft)	Feet &	inches
One Bag	0.242	13.047	2.50	2	6

4.5 Volume Reduction Percentages

The excess thickness before mixing and water addition is due to voids in the aggregate. After mixing and compaction the thickness will be the desired depth. By measuring test specimens in laboratory testing before and after mixing the aggregates, the volume reduction percentage due to mixing alone could be calculated. Using this

reduction percentage, the reduction in the layers as the mixer passes over them can be estimated. This layer thickness reduction allows the mixer to work its way into the layers during mixing. Table 4.14 lists the layer

thickness comparison for each material. An explanation of the column headings is as follows:

1. The pre-mix thickness is the thickness of each layer before mixing.

2. The reduced thickness is the layer thickness after mixing of aggregates with water and vibration for 2 minutes.

3. The reduced fluff is the difference between the previous two measurements for each material.

4. The absolute thickness is based on the absolute volumes within the concrete after the water is added and thoroughly mixed in and the mix is compacted.

5. The absolute fluff is the difference between the absolute thicknesses and the pre-mix thicknesses. The reduction percentages were calculated by the use of equation 4.16.

$$\text{Red\%} = (\text{Pre.Mix} - \text{After.Mix})/\text{Pre.Mix} \qquad (4.16)$$

The reduction percentage (Red%) for a test specimen that measured 8.73 inches before mixing and 8.0 inches after measuring equals 9.3%. Therefore, the aggregate and cement layers were assumed to reduce equally by 9.3% as shown below in Table 4.14.

Table 4.14 Layer Thickness Comparison

	Pre-Mix Thick (in)	Red Thick (in)	Red FLUFF (in)	Absolute Thick (in)	Absolute FLUFF (in)
Cement	0.572	0.519	0.053	0.440	0.132
Coarse	4.505	4.086	0.419	2.940	1.565
Fine	3.592	3.258	0.334	1.823	1.769
Air	0.000	0.000	0.000	0.120	-0.120
Water	0.000	0.000	0.000	0.677	-0.677
Total	8.668	7.863	0.806	6.000	2.668

QUALITY VERIFICATION
CHAPTER 5

5.1 Introduction

Quality verification involved testing the concrete mix for strength at various intervals of time. The two major variables were the method of mixing and the method of compacting. There were two methods of mixing; laboratory mixing and field mixing. Laboratory mixing consisted of using a small mixing drum to thoroughly mix the materials. Field mixing consisted of using soil stabilizing equipment to mix the material laid down in layers of coarse aggregate, cement powder, and fine aggregate. There were two methods of compaction; laboratory compaction and field compaction. Laboratory compaction consisted of using a compaction hammer or a small sledge hammer to strike the mixed material contained in a test mold. Field compaction consisted of the vibratory roller compacting material placed in the test slab filling the test crater.

A third major variable that existed was the method of measuring the quantity of materials put into the mix. The two methods were by weighing and by pre-mix volumes. Test specimens that were laboratory mixed had their materials weighed by scale to measure input quantities. Test specimens that were field mixed had their quantities measured based on pre-mix volumes as discussed in section 4.2. Therefore, laboratory mixing implies input quantities measured by weight and field mixing implies input quantities measured by pre-mix volume. For this reason only two

variable combinations are discussed (mixing and compaction) with the third variable implied (measuring input quantities).

The testing involved test specimens prepared under three different combinations of the two major variables of method of mixing and method of compaction. The first combination was a concrete mix that was laboratory mixed (drum mixer) and laboratory compacted (compaction hammer). The second combination was field mixed (soil stabilizer) and laboratory compacted (small sledge hammer). The third combination was field mixed and field compacted (vibratory roller).

The results were analyzed by using simple linear relationships. Although the linear relationships were not completely accurate, they provided valuable insight as to basic compaction versus strength relationships at the expense of some accuracy.

5.2 Settlement Tests

A settlement test was conducted only on the field mixed and compacted material. The settlement test was begun as soon as the compaction activity was completed. The test consisted of placing a 100 lb plate that was supported by 4 steel rods that provided a contact of 1 square inch all totaled. Figures 5.1 & 5.2 illustrate the plate placement procedure. Therefore, the stress placed on the slab by the 1 steel plate was 100 psi. The stress was increased by placing additional 100 lb plates to a maximum of 4 plates

for a 400 psi stress. Figure 5.3 shows the maximum stress plate arrangement.

Figure 5.1 Placement of Plate Load

Figure 5.2 Load Plate (100 psi)

Figure 5.3 Fully Loaded Load Plate (400 psi)

The placement of the plates after compaction was considered to be time 0. At time 0, settlements occurred as detailed in Table 5.1.

Table 5.1 Settlements in Plate Load Test

Stress (psi)	Rod 1 (in)	Rod 2 (in)	Rod 3 (in)	Rod 4 (in)	Average (in)
100	0.25	0.25	0.0	0.00	0.125
200	0.1	2.00	1.25	1.25	1.150
400	1.75	1.50	1.75	1.50	1.625

At time +1 hour, the settlements were 0 inches at a 400 psi stress.

5.3 Laboratory Testing of Laboratory Mixed and Compacted Test Specimens

The first combination of major variables was material that was laboratory mixed and compacted. Tests on this first variable combination were performed to establish the strengths obtainable based on the mix design only (i.e. eliminate mixing and compaction error). These tests were performed on test specimens prepared by weighing the portions of materials to eliminate proportioning errors. The materials were then mixed using a small concrete drum mixer to provide complete mixing. The mix was placed in test specimen molds and then compacted with a CN-415 Standard Compaction Hammer (5.5 lb * 12 inch drop) to obtain a minimum of 95% compaction. Six 4 inch * 8 inch cylinders and three 3 inch * 3 inch * 11 inch beams were prepared for testing.

The preparation of the test specimens to test the mix design was conducted as to eliminate all other possible causes of error that could cause a weak mix to be developed. Hence, the use of a mixing drum, scale, and compaction hammer were required.

The cylinders were capped in accordance with ASTM C617 (Capping Cylinder Concrete Specimens). The compression tests were performed in accordance with ASTM C39-86 (Test Method for Compressive Strength of Cylindrical Concrete Specimens). The flexural strength tests were performed in accordance with ASTM C78-84 (Test Method for Flexural Strength of Concrete). The tensile strength tests were performed in accordance with ASTM C496-87 (Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens).

The test specimens were tested for compressive, flexural, and tensile strength at 24 hours and 8 days after the mix preparation. The results of the compressive tests had to be corrected due to the shortness of the samples. ASTM C42-85 (Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete) specifies that samples with a length to diameter ratio (L/d) below 1.94 must have correction factors applied to reduce strengths obtained (30). The stresses were calculated by dividing the load by the area of the cylinder. Table 5.2 lists the cylinders along with the loads, areas, uncorrected stresses,

correction factors and corrected compressive stresses.

Table 5.2 Laboratory Cylinder Corrected Compressive Stress

Cylinder	Load (lbs)	Area (in^2)	Init. Comp. Stress (psi)	Red. Factor (%)	Corr. Comp. Stress (psi)
1C	39490	12.566	3140	0.9600	3010
2C	44650	12.566	3550	0.9300	3300
3C	51230	12.566	4080	0.9600	3920
4C	65010	12.566	5170	0.9600	4960

The unit weights of the samples were calculated by dividing the weight of a sample by the volume of the sample. The degree of compaction was measured based on the mix design unit weight. The maximum compaction obtainable was 100% which allowed for zero air voids (ZAV). The mix design produced a mix with a unit weight of 142.3 pcf assuming 2% entrapped air. The magnitude of compaction was defined as in equation 5.1.

$$\text{Compaction} = \frac{\text{Unit Wt}}{\text{Unit Wt. @ ZAV}} \qquad (5.1)$$

The unit weight for ZAV was obtained as shown in equation 5.2.

@ 2% Air Voids, Unit Weight = 142.3 pcf

Compaction = 98.0%

$$\text{Unit Wt. @ ZAV} = \frac{142.3 \text{ pcf}}{0.98} \qquad (5.2)$$

Unit Wt. @ ZAV = 142.3 pcf

Compaction = 100%

Table 5.3 lists the unit weights, compaction %'s, corrected compressive stresses, and curing times of each cylinder.

Table (5.3) Laboratory Cylinder Compressive Strength Data Sheet

Cylinder	Unit Wt. (pcf)	Comp. (%)	Corrected Compress Stress (psi)	Curing Time (hrs)
1C	137.4	94.6%	3010	24
2C	144.1	99.3%	3300	24
3C	138.0	95.0%	3920	192
4C	142.6	98.2%	4960	192

Two cylinders were tested for tensile stress failure. The maximum stresses were calculated using equation 5.3 (31).

T = Splitting Tensile Strength

P = Maximum Applied Load

l = length

d = diameter

$$T = (2 * P)/(pi * l * d) \qquad (5.3)$$

The splitting tensile test results are listed in Tables 5.4 & 5.5.

Table 5.4 Laboratory Tensile Cylinder Unit Weight and Compaction

Cylinder	Diam. (in)	Len. (in)	Vol. (cf)	Wt. (lbs)	Unit Wt. (pcf)	Comp. (%)
1T	4	6.23	0.0453	6.32	139.5	96.1%
2T	4	6.13	0.0445	6.21	139.4	96.0%

Table 5.5 Laboratory Tensile Cylinder Strength

Cylinder	Load (lbs)	T (psi)	Curing Time (hrs)
1T	13870	355	24
2T	9370	243	192

Cylinder #2T had significant honeycombing concentrated at one end.

Three beams were tested for flexural strength. The equations used to calculate the flexural stress were equations 5.4, 5.4a, and 5.4b (32).

R = Flexural Stress = Modulus of Rupture

M = Maximum Bending Moment

c = One-Half the Depth of the Beam

I = Moment of Inertia of the Cross Section

P = Load

L = Length Under Load

d = Depth of Beam

$$R = (M * c)/I \qquad (5.4)$$

$$M = (P * L)/4 \qquad (5.4a)$$

$$I = (b * d3)/12 \qquad (5.4b)$$

Tables 5.6, 5.7 and 5.8 list the results of flexural tests.

Table 5.6 Laboratory Flexural Beam Dimensions and Unit Weight

Beam #	Actual Len. (in)	Width (in)	Depth (in)	Vol. (cf)	Wt. (lbs)	Unit Wt. (pcf)
1B	11	2.48	3.00	0.0473	6.87	145.2
2B	11	2.48	3.00	0.0473	6.69	141.5
3B	11	2.50	3.00	0.0478	6.94	145.2

Table 5.7 Laboratory Beam Flexural Stress (R)

Beam #	Total Load (lbs)	Loaded Length (in)	Neutral Axis c (in)	Moment of Inertia (in^4)	Comp. (%)	R (psi)
1B	1340	9	1.5	5.578	100.0%	811
2B	1380	9	1.5	5.569	97.4%	836
3B	1730	9	1.5	5.630	100.0%	1037

Table 5.8 Curing Time

Beam #	Curing Time (hrs)
1B	24
2B	192
3B	192

The results of the tests will be discussed in section 5.6 which compares the test results.

The laboratory testing provided a benchmark for the

judging the compressive strengths obtained in the field test. The mix design detailed in section 3.2 was followed to produce the laboratory test specimens.

5.4 Laboratory Testing of Field Mixed and Laboratory Compacted Test Specimens

The second combination of field mixed and laboratory compacted was performed to minimize compaction error and isolate the effect of strengths by field mixing.

After completion of the mixing operation by the soil stabilizer during the field test, samples were removed from the mixed material and placed into cylinders. Figure 5.4 shows the mixed material in place that the sample material was taken from. The cylinders were placed on the vibratory drum and vibrated for four minutes as shown in figure 5.5. The cylinders were then compacted using a small sledge hammer and a small tamper as shown in figure 5.6. Seven 6 inch * 12 inch cylinders were prepared for compression tests. The flexural and tensile strengths were obtained by using known flexural and tensile strength relationships for concrete (33). Figure 5.7 shows two of the sawcut cylinders. Figure 5.8 shows the capping procedure being performed on a cylinder in accordance with ASTM C617 (capping Cylinder Concrete Specimens).

Figure 5.4 Mixed, Uncompacted Slab

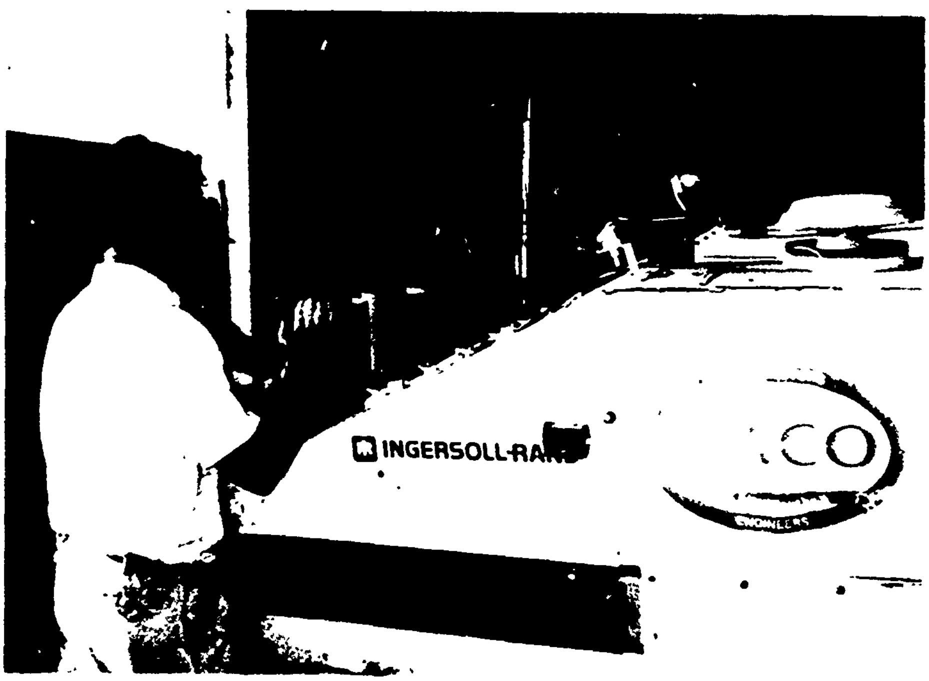


Figure 5.5 Cylinder Vibration

Figure 5.6 Cylinder Compaction

Figure 5.7 Sawcut Test Cylinders

Figure 5.8 Cylinder and Cylinder Capping Mold

Figure 5.9 shows the cylinders undergoing the compression test in accordance with ASTM C39-86 (Test Method for Compressive Strength of Cylindrical Concrete Specimens).

Table 5.9 lists the results obtained from the compression tests including corrected stresses.

Table 5.9 Field Cylinder Corrected Compressive Stresses

Cylinder	Load (lbs)	Area (in^2)	Init. Comp. Stress (psi)	Red. Factor (%)	Corr. Comp. Stress (psi)
5C	42500	28.274	1500	0.9445	1420
6C	41500	28.274	1470	0.9560	1410
7C	48250	28.274	1710	0.9445	1620
8C	47750	28.274	1690	0.9600	1620
9C	42000	28.274	1490	0.9670	1440
10C	77250	28.274	2730	0.9660	2640
11C	79500	28.274	2810	0.9700	2730

Figure 5.9 Compressive Test Performance

Table 5.10 lists the compaction percentages, corrected stresses and curing times side by side.

Table 5.10 Field Cylinder Compressive Strength Data Sheet

Cylinder	Unit Wt. (pcf)	Comp. (%)	Corrected Compress Stress (psi)	Curing Time (hrs)
5C	141.7	97.6%	1420	14
6C	141.7	97.6%	1410	14
7C	141.1	97.2%	1620	14
8C	140.5	96.8%	1620	14
9C	140.5	96.8%	1440	24
10C	141.2	97.2%	2640	24
11C	140.1	96.5%	2730	72

The results of the tests will be discussed in section 5.6 which compares the test results.

5.5 Laboratory Testing of Field Mixed and Compacted Test Specimens

The field mixed and compacted samples were obtained the day following the slab construction. The samples were actually cored out of the slab using a coring machine. Since the samples had to be field mixed by the mixer and field compacted by the vibratory roller, obtaining the core samples was the only method of producing test specimens. Six core samples were taken from different locations throughout the slab. Appendix D lists the exact locations in the slab that the core samples were taken from. The

flexural and tensile strengths were obtained by using known flexural and tensile strength relationships for concrete (34).

Table 5.11 lists the results obtained from the core samples.

Table 5.11 Field Core Corrected Compressive Stresses

Core	Load (lbs)	Area (in^2)	Init. Comp. Stress (psi)	Red. Factor (%)	Corr. Comp. Stress (psi)
12C	21400	6.026	3550	1.0000	3550
13C	10000	6.02	1660	1.0000	1660
14C	15000	6.026	2490	1.0000	2490
15C	14400	6.026	2390	1.0000	2390

Table 5.12 lists compaction percentages side by side with the corrected compressive stresses and curing time of each sample.

Table 5.12 Compressive Strength Data Sheet

Core	Unit Wt. (pcf)	Comp. (%)	Corrected Compress Stress (psi)	Curing Time (hrs)
12C	141.3	97.3%	3550	72
13C	133.6	92.0%	1660	72
14C	140.5	96.8%	2490	168
15C	138.6	95.5%	2390	168

The results of the tests will be discussed next in section 5.6.

5.6 Comparison of Laboratory Mixed and Compacted, Field Mixed and Laboratory Compacted, and Field Mixed and Compacted Test Specimens

The results of each combination tested are listed in Table 5.13 below.

Table 5.13 Summary of All Actual Stresses

SPECIMEN #	COMPRESS (psi)	FLEX (R) (psi)	TENSILE (T) (psi)	COMPACT'N (%)	TIME (HRS)
1C	3010			94.6%	24
2C	3300			99.3%	24
3C	3920			95.0%	192
4C	4960			98.2%	192
5C	1420			97.6%	14
6C	1410			97.6%	14
7C	1620			97.2%	14
8C	1620			96.8%	14
9C	1440			96.8%	24
10C	2640			97.2%	24
11C	2730			96.5%	72
12C	3550			97.3%	72
13C	1660			92.0%	72
14C	2490			96.8%	168
15C	2390			95.5%	168
1B		811		100.0%	24
2B		836		97.4%	192
3B		1037		100.0%	192
1T			355	96.1%	24
2T			243	96.0%	192

Since each sample could be tested for only one type of stress (compressive, tensile, or flexural) known relationships for compressive to flexural, compressive to tensile, and tensile to flexural were used to obtain a set of three stresses for each sample (35). The values of the stress relationships used are listed in Table 5.14.

Table 5.14 Stress Relationships

Split Tensile/Compressive	= 10.0%
Flexural/Compressive	= 15.0%
Split Tensile/Flexural	= 62.5%

Therefore, to obtain a flexural value for specimen 1C, the compressive strength (stress) value was multiplied by the Flexural/Compressive ratio to obtain a flexural value. The laboratory mixed and compacted sample relationships of flexural to compressive (approx. 26%) and tensile to compressive (11%) exceed the relationships in Table 5.14 Later calculations developed in this report are based on compressive strengths. Therefore, the lower ratios in Table 5.14 are conservative since they will produce lower compressive strengths from flexural and tensile strengths as tabulated in Table 5.15.

Table 5.15 Actual and Calculated Stress Values

SPECIMEN #	COMPRESS (psi)	FLEX (R) (psi)	TENSILE (T) (psi)	COMPACT'N (%)	TIME (HRS)
1C	3010	452	301	94.6%	24
2C	3300	495	330	99.3%	24
3C	3920	588	392	95.0%	192
4C	4960	744	496	98.2%	192
5C	1420	213	142	97.6%	14
6C	1410	212	141	97.6%	14
7C	1620	243	162	97.2%	14
8C	1620	243	162	96.8%	14
9C	1440	216	144	96.8%	24
0C	2640	396	264	97.2%	24
11C	2730	410	273	96.5%	72
12C	3550	533	355	97.3%	72
13C	1660	249	166	92.0%	72
14C	2490	374	249	96.8%	168
15C	2390	359	239	95.5%	168
1B	5405	811	507	100.0%	24
2B	5576	836	523	97.4%	192
3B	6914	1037	648	100.0%	192
1T	3546	567	355	96.1%	24
2T	2435	390	243	96.0%	192

5.6.1 Compressive Strength Versus Compaction%

The relationship of compaction percentage versus compressive strength was investigated.

Th laboratory mixed and compacted samples tested at 24 hours (1C, 2C, 1B, 1T) yielded substantial scatter of compressive strengths. The results were processed through linear regression to obtain a simplified linear relationship. The linear relationship shows that the strength increased as compaction increased. Figure 5.10 shows the points and linear relationship.

The laboratory mixed and compacted samples that were tested at 8 days (3C, 4C, 2B, 3B, 2T) yielded a minimum of scatter for compressive strengths. The results were processed through linear regression to obtain a simplified linear relationship. The linear relationship shows that the strength increased as the compaction increased. Figure 5.11 shows the points and linear relationship.

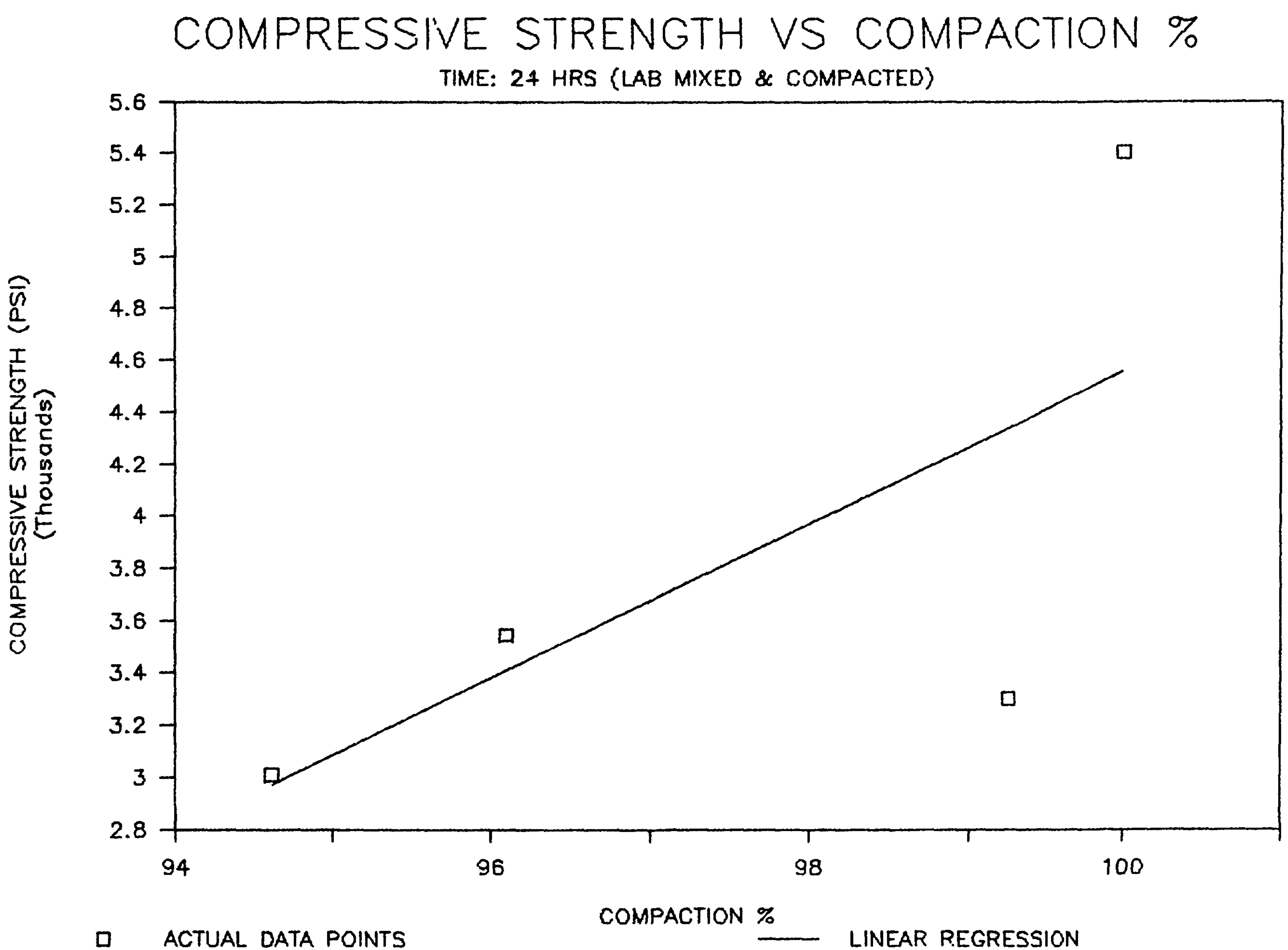
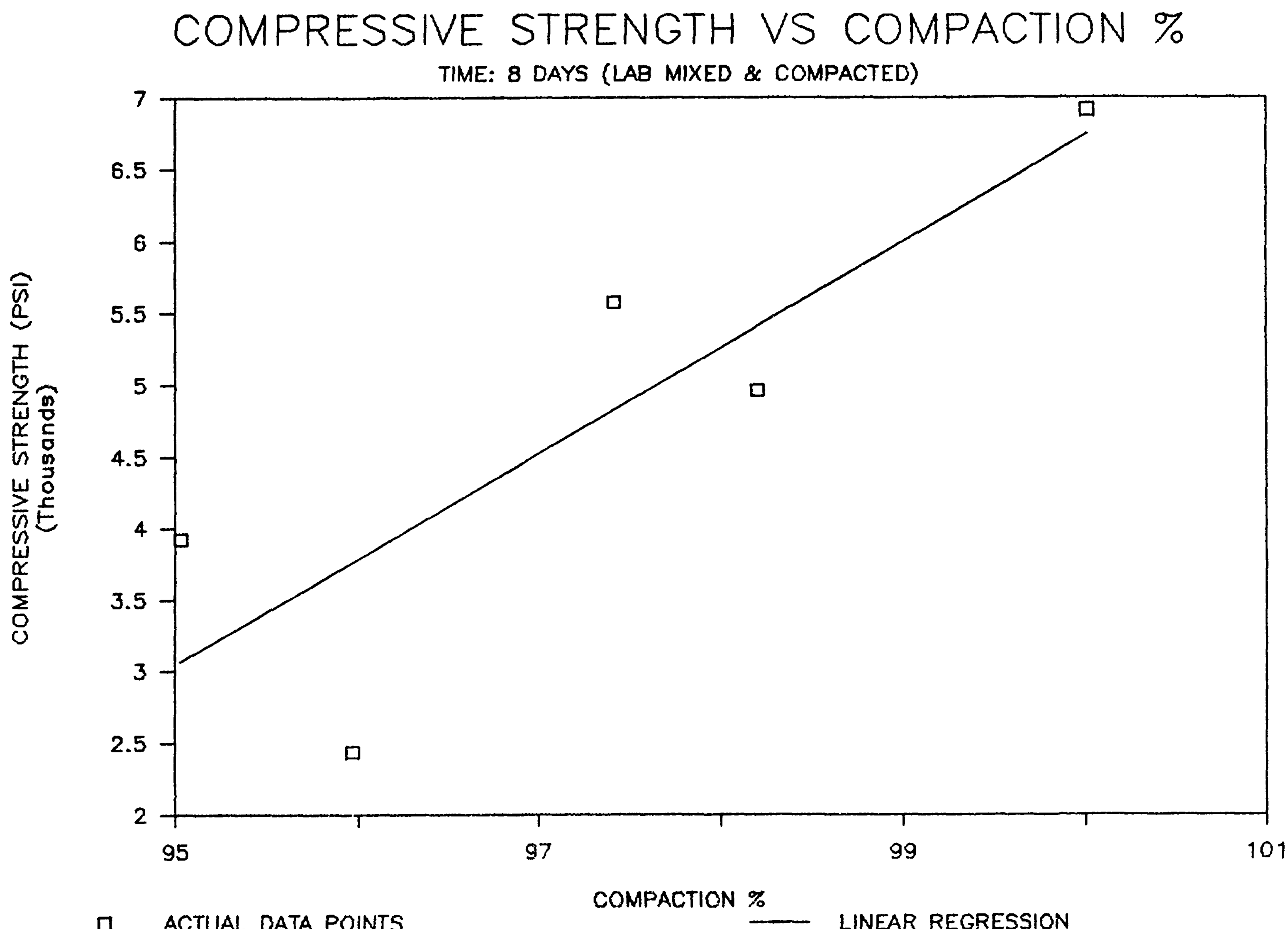

COMPRESSIVE STRENGTH VS COMPACTION %
TIME: 24 HRS (LAB MIXED & COMPACTED)
COMPRESSIVE STRENGTH (PSI)
(Thousands)
5.6
5.4
5.2
5
4.8
4.6
4.4
4.2
4
3.8
3.6
3.4
3.2
3
2.8
94
96
98
100
COMPACTION %
ACTUAL DATA POINTS
LINEAR REGRESSION

Figure 5.10



COMPRESSIVE STRENGTH VS COMPACTION %
TIME: 8 DAYS (LAB MIXED & COMPACTED)
COMPRESSIVE STRENGTH (PSI)
(Thousands)
7
6.5
6
5.5
5
4.5
4
3.5
3
2.5
2
95
97
99
101
COMPACTION %
ACTUAL DATA POINTS
LINEAR REGRESSION

Figure 5.11

The field mixed and laboratory compacted samples tested at 14 hours (5C, 6C, 7C, 8C) yielded a very small amount of scatter of compressive strengths. The results were processed through linear regression to obtain a simplified linear relationship. The linear relationship shows that the strength slightly decreased as compaction increased. The closeness of the compaction percentages (96.8% to 97.6%) idicates that the compaction increases from sample to sample were not significant enough to overcome other factors such as poor mixing that could cause small drops in compressive strength. Figure 5.12 shows the points and the linear relationship.

The field mixed and laboratory compacted samples tested at 24 hours (9C, 10C) yielded a very large amount of difference of compressive strengths. The simplified linear elationship shows that the strength greatly increased as compaction increased. The closeness of the compaction perentages (96.8% to 97.2%) indicates that the compaction increases from sample to sample were not significant enough to cause the large increases in compressive strength. The combination of lower compaction and other factors such as poor mixing could hav contributed to the low strength of sample 9C. Figure 5.13 shows the points and the linear relationship.

COMPRESSIVE STRENGTH VS COMPACTION %

TIME: 14 HRS (FIELD MIX, LAB COMPACTED)

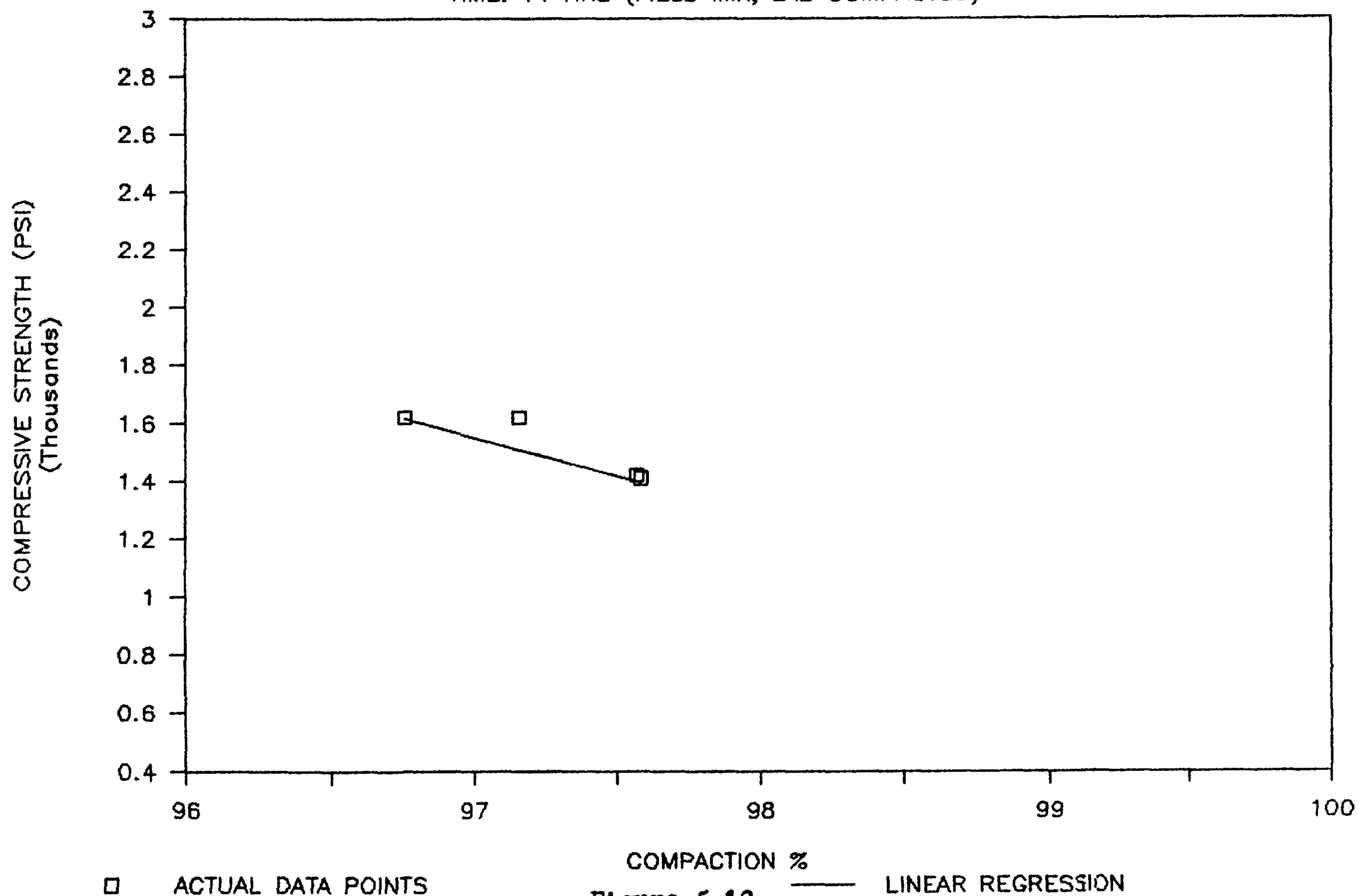
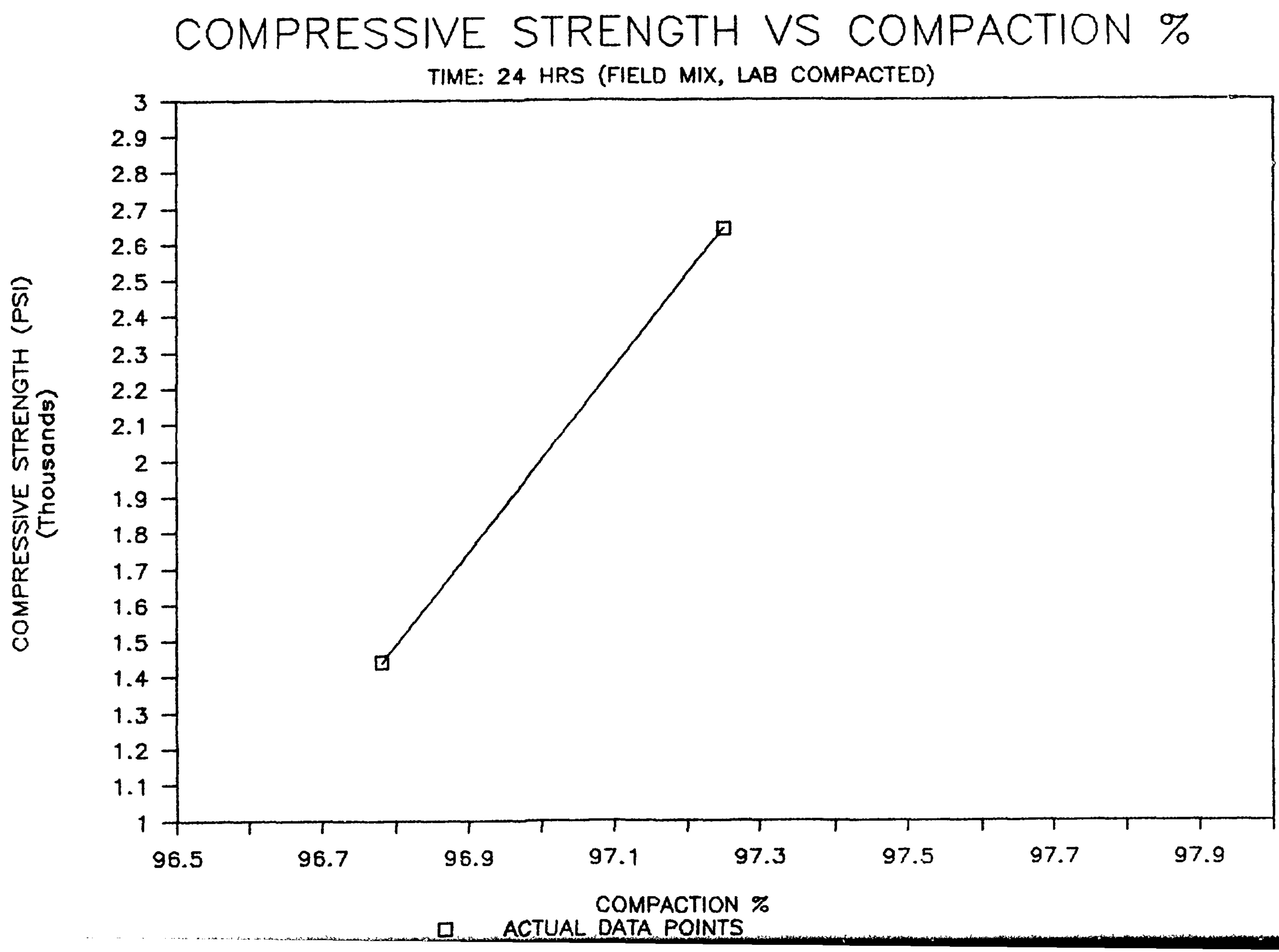
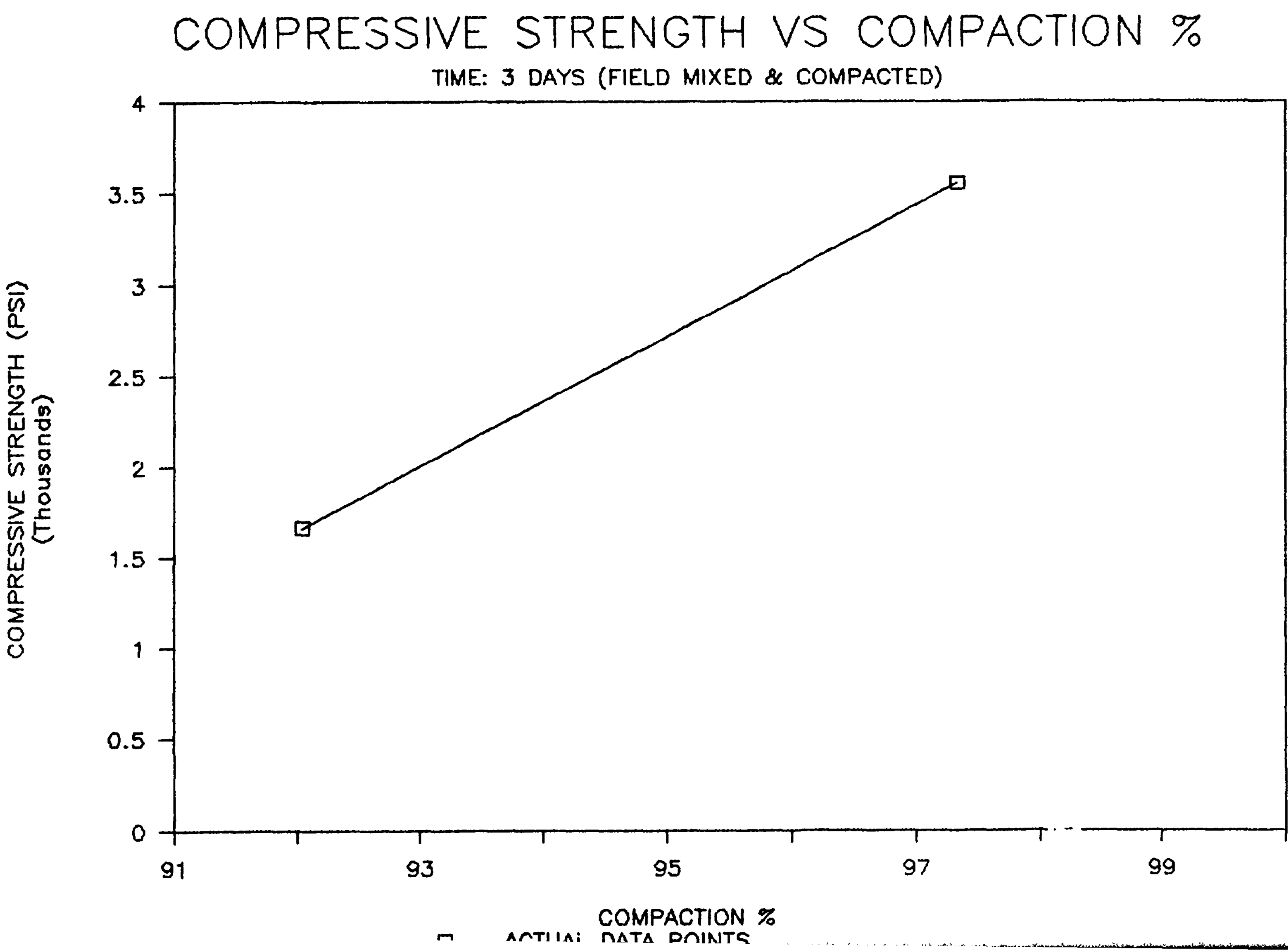


Figure 5.12


COMPRESSIVE STRENGTH VS COMPACTION %
TIME: 24 HRS (FIELD MIX, LAB COMPACTED)
COMPRESSIVE STRENGTH (PSI)
(Thousands)
3
2.9
2.8
2.7
2.6
2.5
2.4
2.3
2.2
2.1
2
1.9
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1
96.5
96.7
96.9
97.1
97.3
97.5
97.7
97.9
COMPACTION %
ACTUAL DATA POINTS

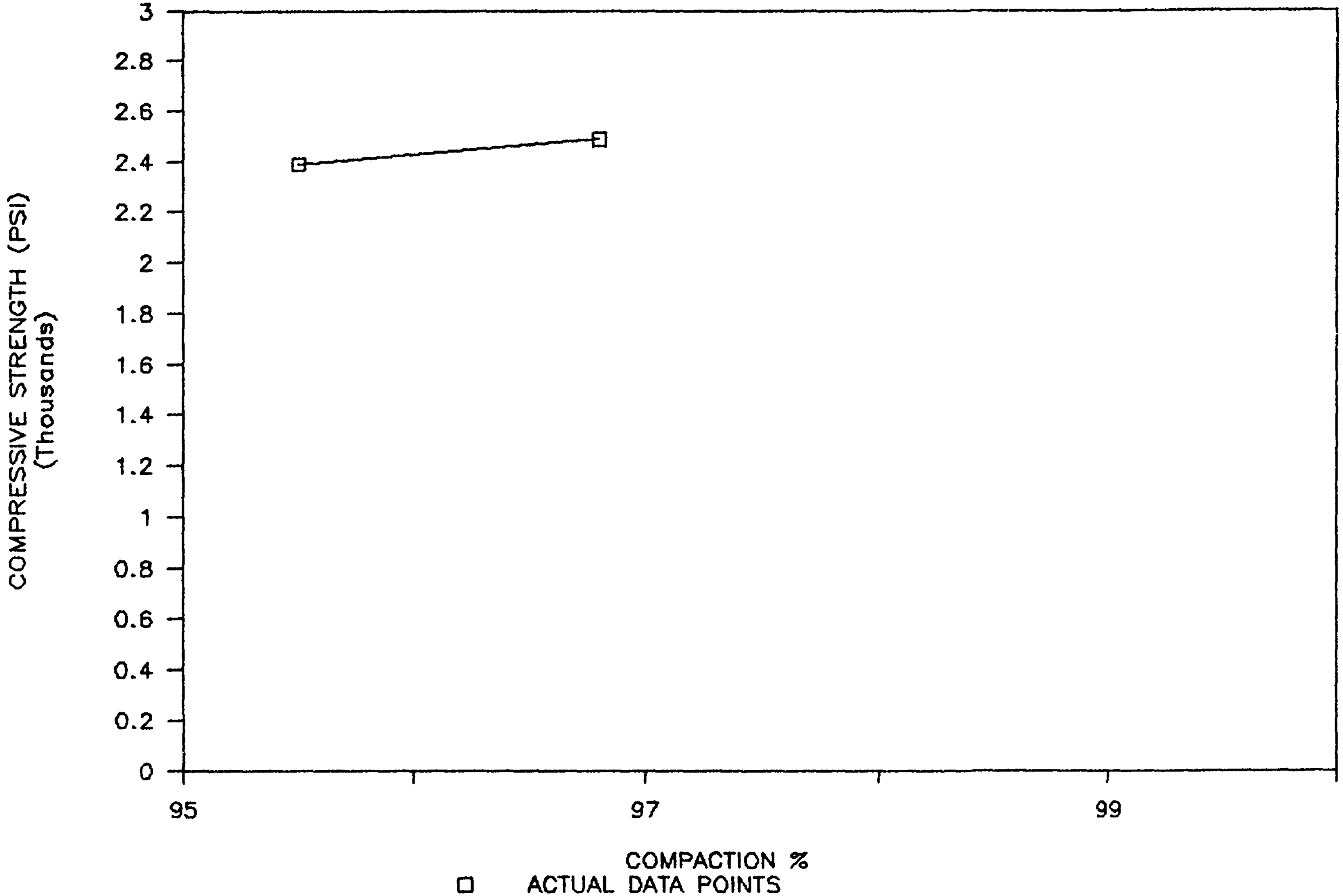
The field mixed and compacted samples tested at 72 hours (12C, 13C) yielded a very large amount of difference of compressive strengths. The simplified linear relationship shows that the strength greatly increased as compaction greatly increased. The large difference of the compaction percentages (92.0% to 97.3%) indicates that the compaction increases from sample to sample were significant enough to cause large increases in compressive strength. However, the excessively high strength may also be affected by improper mixing causing concentrations of cement powder. Figure 5.14 shows the points and the linear relationship.

The field mixed and compacted samples tested at 168 hours (14C, 15C) yielded a very small amount of difference of compressive strengths. The linear relationship shows that the strength slightly increased as compaction slightly increased. There was a small difference of the compaction percentages (95.5% to 96.8%). Figure 5.15 shows the points and the linear relationship.


COMPRESSIVE STRENGTH VS COMPACTION %
TIME: 3 DAYS (FIELD MIXED & COMPACTED)
COMPRESSIVE STRENGTH (PSI)
(Thousands)
4
3.5
3
2.5
2
1.5
1
0.5
0
91
93
95
97
99
COMPACTION %

COMPRESSIVE STRENGTH VS COMPACTION %

TIME: 7 DAYS (FIELD MIXED & COMPACTED)



5.6.2 Compressive Strength Versus Time

The mean compressive stress for each time interval was obtained by averaging the strengths of the cylinders tested at the particular time interval. The variance (Var.) shows a the percentage variation of each cylinder from the mean. The amount (Amt.) was the magnitude in lbs that each sample differed from the mean. Table 5.16 lists the aforementioned parameters for the compressive cylinders along with the method each cylinder was mixed and compacted by.

Table 5.16 Mean Compressive Stresses

Cyl.	Curing Time (hrs)	Time (days)	Mix Method	Comp. Method	Mean Compress Stress (psi)	Var. (%)	Amt. (lbs)
1C	24	1.0	LAB	LAB	3155	-4.6%	-145
2C	24	1.0	LAB	LAB	3155	4.6%	45
3C	192	8.0	LAB	LAB	4440	-11.7%	-520
4C	192	8.0	LAB	LAB	4440	11.7%	520
5C	14	0.6	FIELD	LAB	1518	-6.4%	-98
6C	14	0.6	FIELD	LAB	1518	-7.1%	-108
7C	14	0.6	FIELD	LAB	1518	6.8%	103
8C	14	0.6	FIELD	LAB	1518	6.8%	103
9C	24	1.0	FIELD	LAB	2040	-29.4%	-600
10C	24	1.0	FIELD	LAB	2040	29.4%	600
11C	72	3.0	FIELD	LAB	N/A	N/A	N/A
Core							
12C	72	3.0	FIELD	FIELD	2605	36.3%	945
13C	72	3.0	FIELD	FIELD	2605	-36.3%	-945
14C	168	7.0	FIELD	FIELD	240	2.0%	50
15C	168	7.0	FIELD	FIELD	2440	-2.0%	-50

The scatter of compaction percentages for the lab mixed and compacted samples was well-distributed. Figure 5.16 shows the average compaction percentage as one line and each sample's compaction percentage. The left four points (1C, 1T, 2C, 1B) are from the one-day compression tests, the remaining five right points (3C, 2T, 2B, 4C, 3B) are from the eight-day compression tests

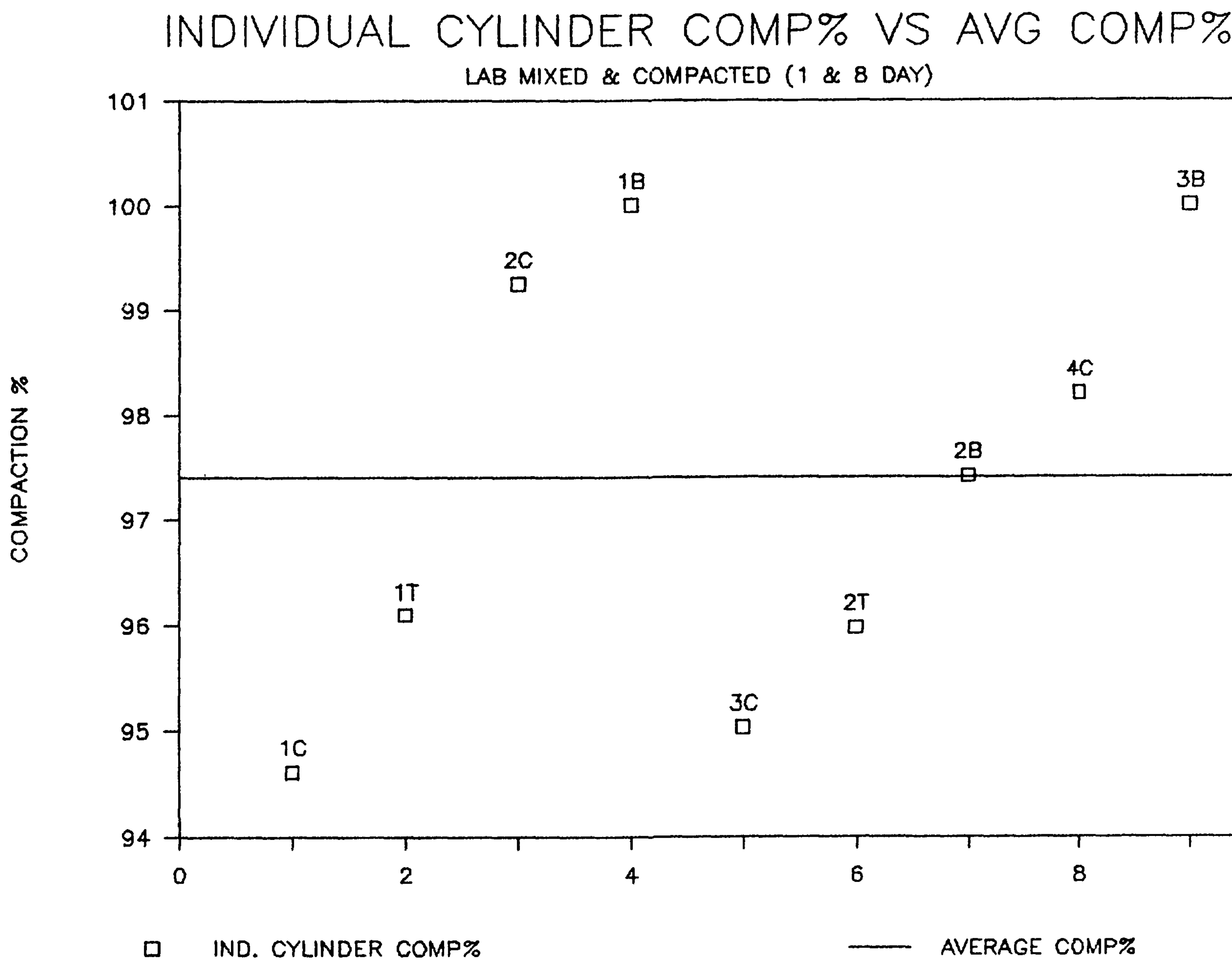

INDIVIDUAL CYLINDER COMP% VS AVG COMP%
LAB MIXED & COMPACTED (1 & 8 DAY)
COMPACTION %
101
100
99
98
97
96
95
94
0
2
4
6
8
10
1C
1T
2C
1B
3C
2T
2B
4C
3B
IND. CYLINDER COMP%
AVERAGE COMP%

Figure 5.16

The average compaction percentages are very close to the overall compaction average as shown in Table 5.17.

Table 5.17 Compaction and Compressive Strength Averages (Lab Mixed and Compacted)

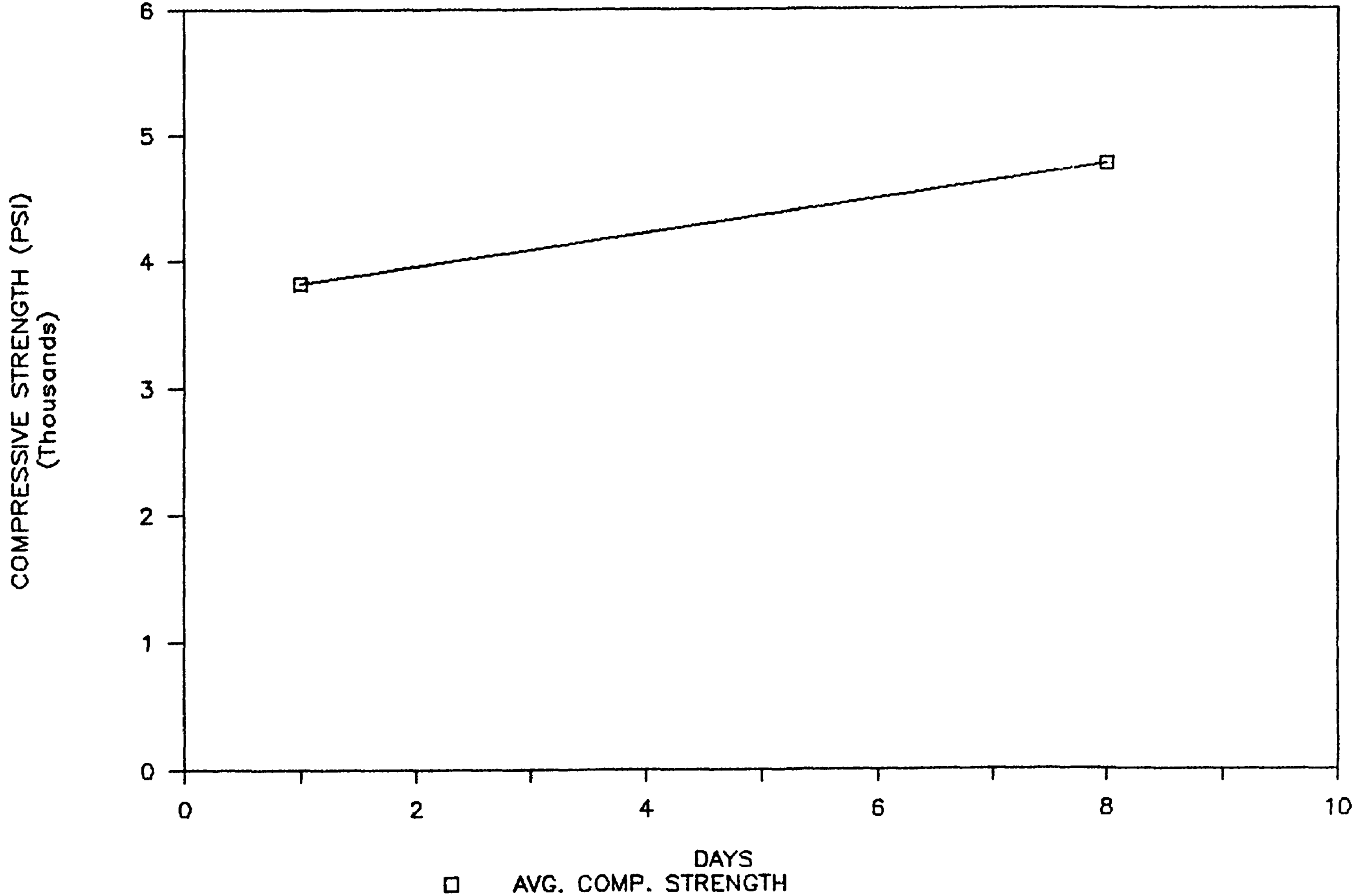
Curing Time (days)	Average Compaction %	Compressive Strength (psi)
1	97.5	3820
8	97.3	4760
Overall	97.4	

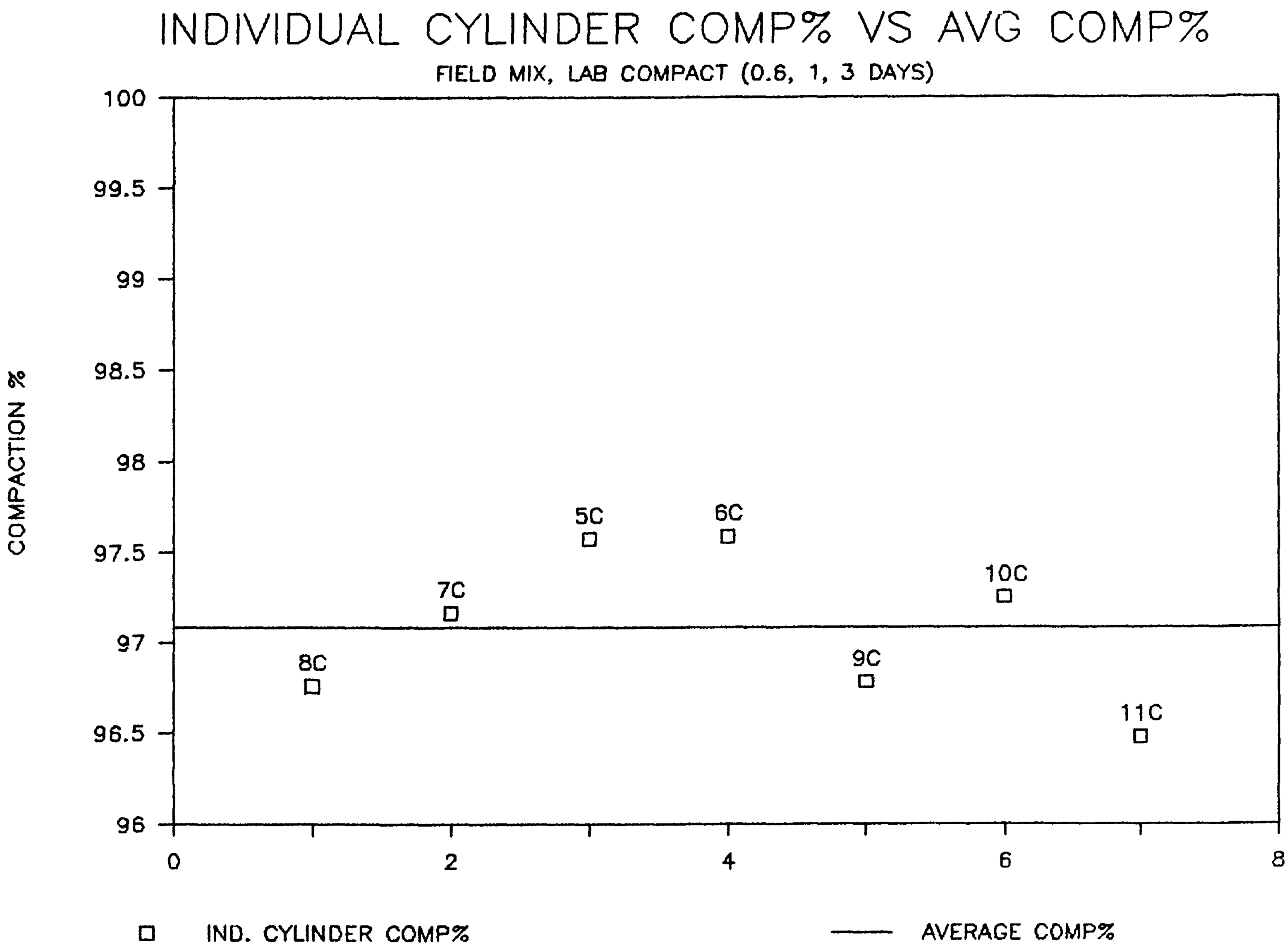
Figure 5.17 shows average compressive strengths versus the time of curing relationship for the lab mixed and compacted samples. The compressive strength increases over time as expected.

The scatter of compaction percentages for the field mixed and laboratory compacted samples was minimal. Figure 5.18 shows the average compaction percentage as one line and each sample's compaction percentage. The left four points (8C, 7C, 5C, 6C) are from the 14-hour (0.6 days) compression test, the next two points (9C, 10C) are from one day compression test and the rightmost point (11C) is from the three day compression test.

COMPRESSIVE STRENGTH VS TIME (DAYS)

LAB MIX & COMPACT (1, 8 DAYS)





INDIVIDUAL CYLINDER COMP% VS AVG COMP%
FIELD MIX, LAB COMPACT (0.6, 1, 3 DAYS)
COMPACTION %
100
99.5
99
98.5
98
97.5
97
96.5
96
0
2
4
6
8
8C
7C
5C
6C
9C
10C
11C
IND. CYLINDER COMP%
AVERAGE COMP%

Figure 5.18

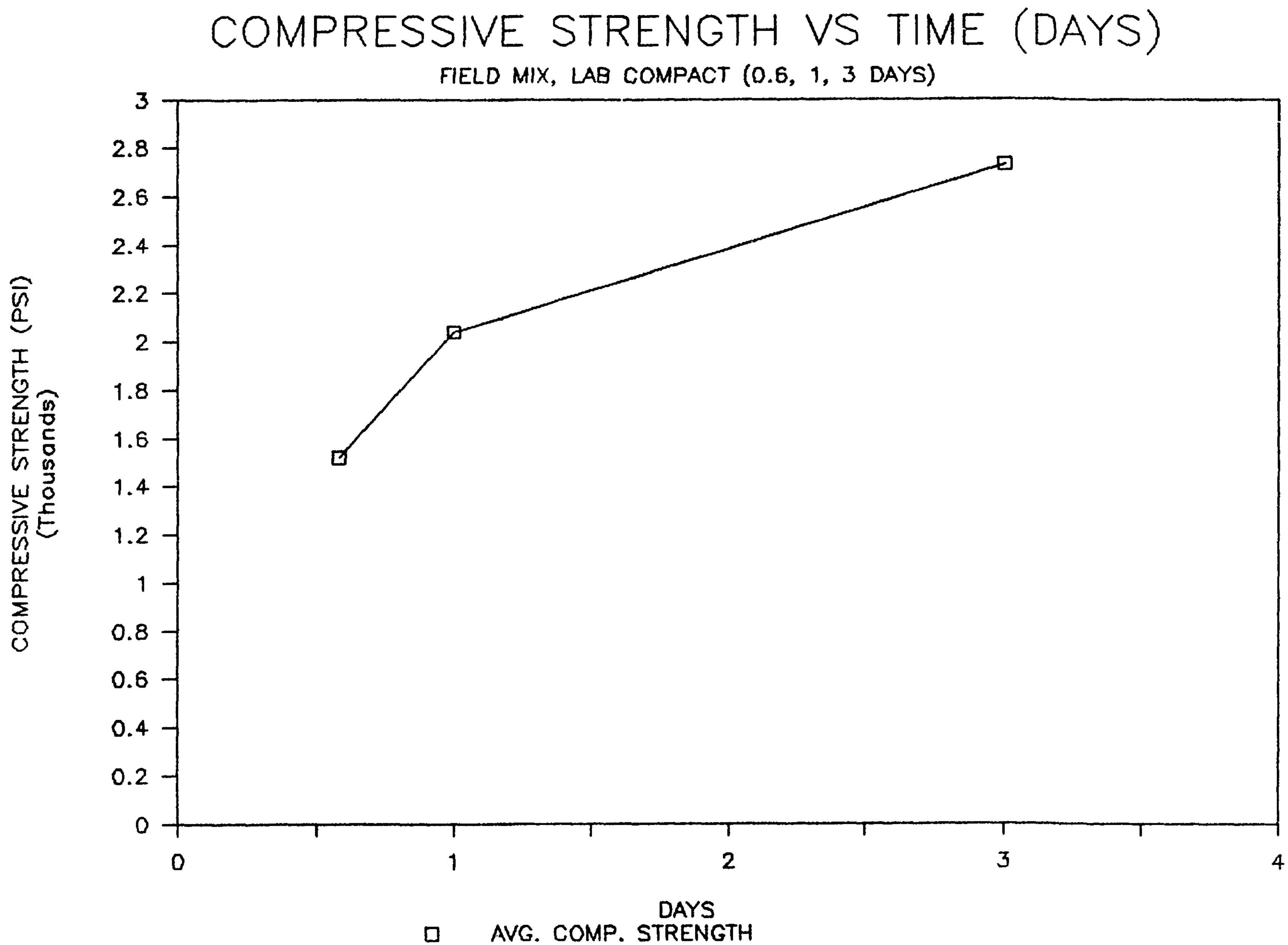
The average compaction percentages are very close to the overall compaction average as shown in Table 5.18.

Table 5.18 Compaction and Compressive Strength Averages (Field Mixed and Lab Compacted)

Curing Time (days)	Average Compaction %	Compressive Strength (psi)
0.6	97.3	1520
1	97.0	2040
3	96.5	2730
Overall	97.4	

Figure 5.19 shows average compressive strengths versus the time of curing relationship for the field mixed and laboratory compacted samples. The compressive strength increases over time as expected.

The scatter of compaction percentages for the field mixed and compacted samples was large and maldistributed. Figure 5.20 shows the average compaction percentage as one line and each sample's compaction percentage. The left two points (12C, 13C) are from the three-day compression tests. The right two points (14C, 15C) are from the seven-day compression tests.



COMPRESSIVE STRENGTH VS TIME (DAYS)
FIELD MIX, LAB COMPACT (0.6, 1, 3 DAYS)
COMPRESSIVE STRENGTH (PSI)
(Thousands)
3
2.8
2.6
2.4
2.2
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
1
2
3
4
DAYS
AVG. COMP. STRENGTH

Figure 5.19

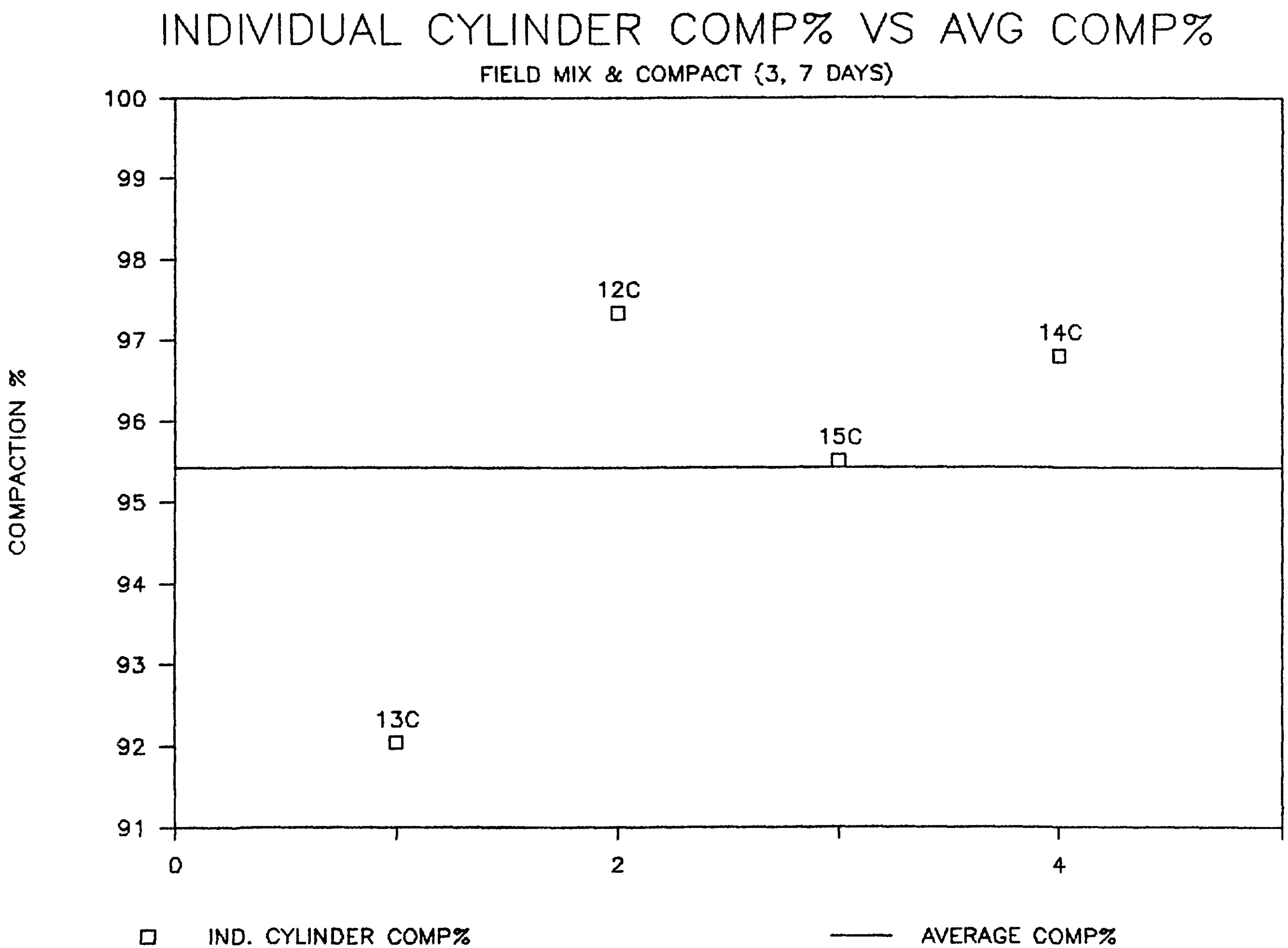

INDIVIDUAL CYLINDER COMP% VS AVG COMP%
FIELD MIX & COMPACT (3, 7 DAYS)
COMPACTION %
100
99
98
97
96
95
94
93
92
91
0
2
4
12C
14C
15C
13C
IND. CYLINDER COMP%
AVERAGE COMP%

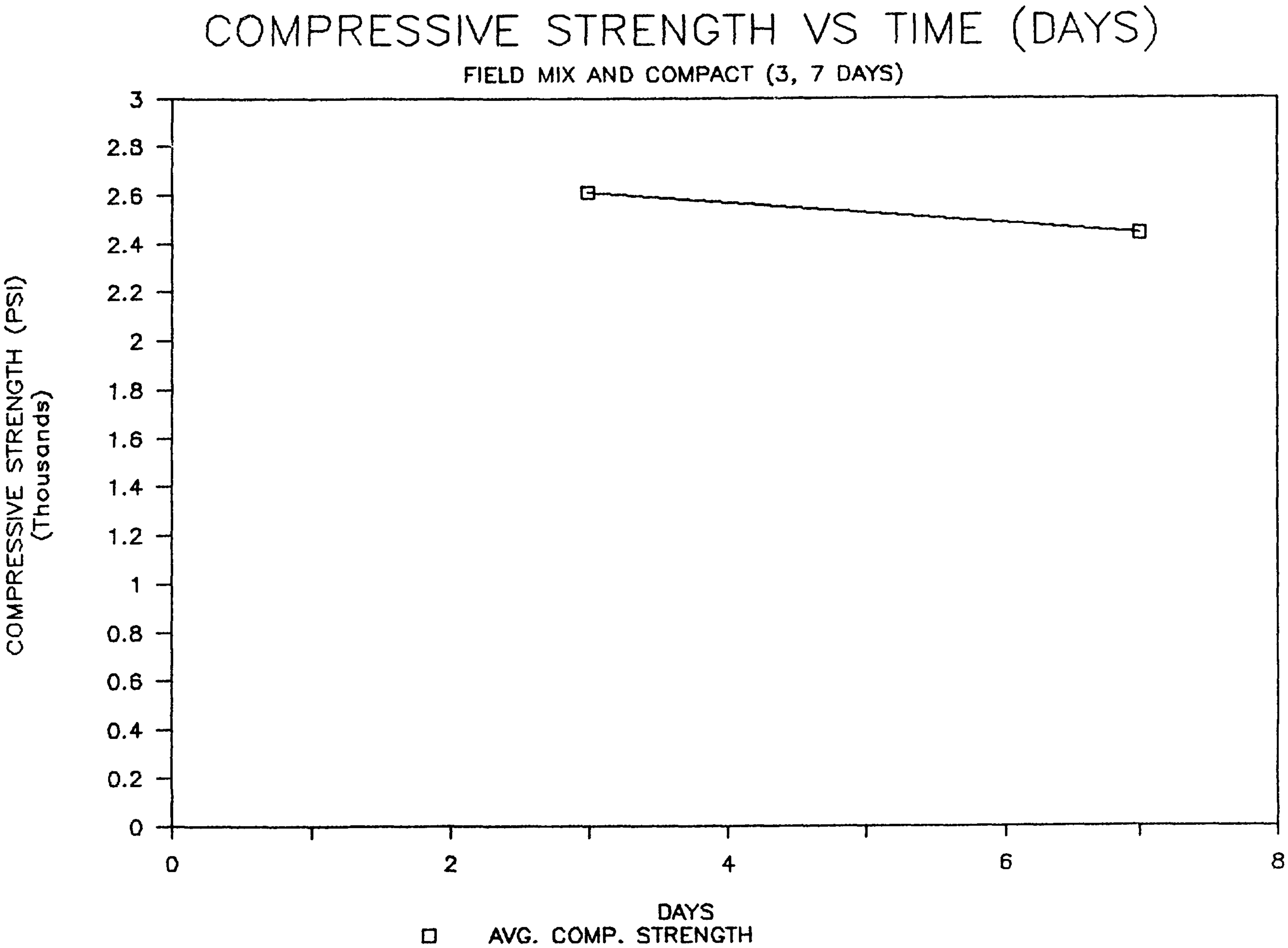
Figure 5.20

In contrast, the average compaction percentages for each test are close to the overall compaction average of all cores as shown in Table 5.19.

Table 5.19 Compaction and Compressive Strength Averages (Field Mixed and Compacted)

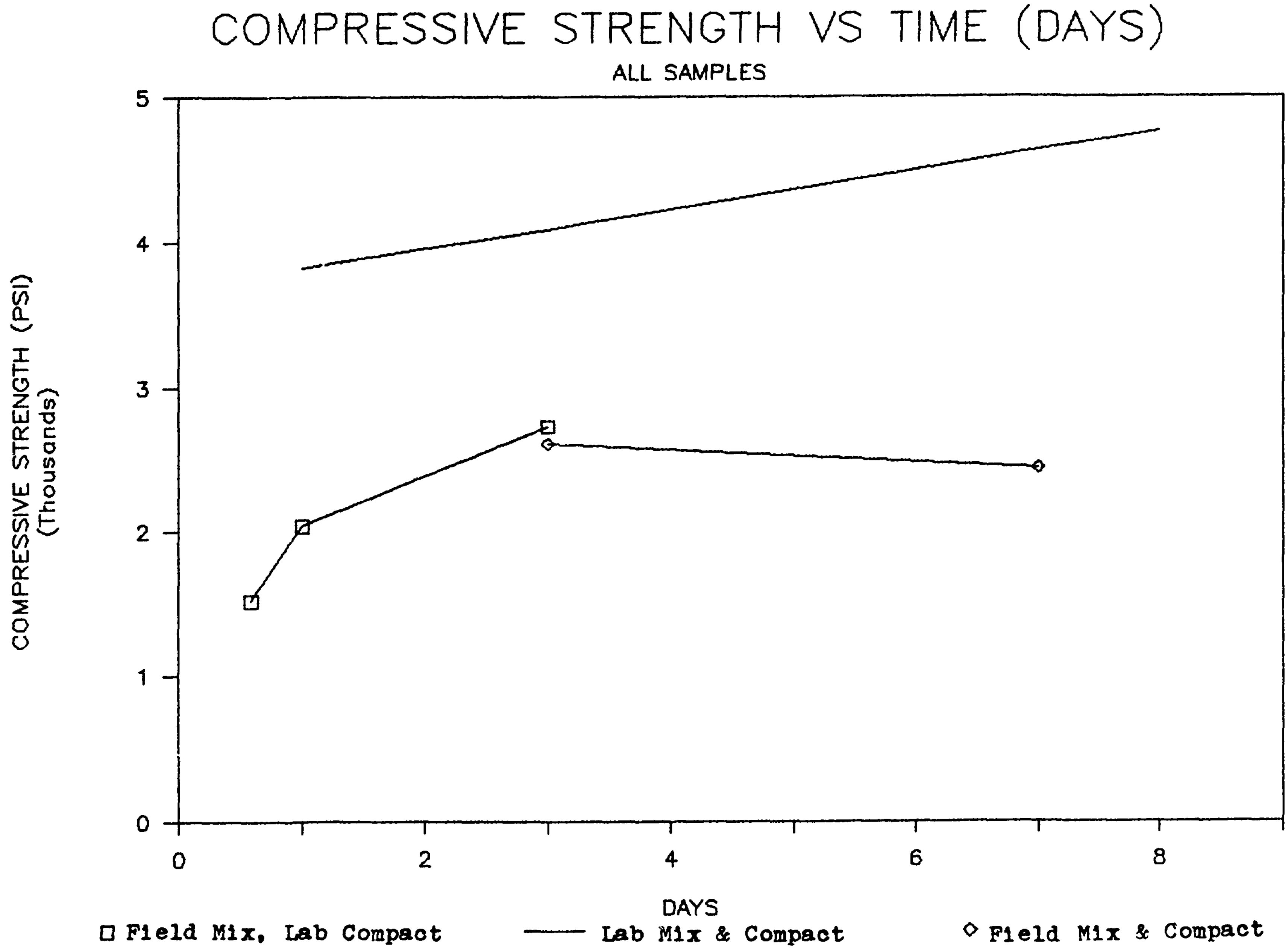
Curing Time (days)	Average Compaction %	Compressive Strength (psi)
3	94.7	2610
7	96.2	2440
Overall	95.4	

Figure 5.21 shows average compressive strengths versus the time c' curing relationship for the field mixed and compacted samples. The high strength of cylinder 12C at 3550 psi, greatly increased the average compressive strength of the three-day test. The use of high early strength cement typically produces approximately 75% of the 28-day strength at the three-day point (36). The strength typically increases to 84% by the seven-day point (37). The seven-day compressive strengths were clustered around their strength of 2440 psi creating an apparent 6.5% decrease in strength from the three-day test average of 2610 psi. Further discussion is presented in the conclusions concerning this apparent erroneous result of decreasing strength over time.



COMPRESSIVE STRENGTH VS TIME (DAYS)
FIELD MIX AND COMPACT (3, 7 DAYS)
COMPRESSIVE STRENGTH (PSI)
(Thousands)
3
2.8
2.6
2.4
2.2
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
2
4
6
8
DAYS
AVG. COMP. STRENGTH

Figure 5.22 shows the three compressive strength versus time graphs superimposed upon one another. The ideal laboratory mixed and compacted graph shows the expected strength increase and overall higher strengths due to a lack of error in material input, mixing, water addition, and compaction. The field mixed and laboratory compacted graph shows the expected increasing strength over time relationship. The strengths are less than obtained in ideal laboratory mixing conditions. The field mixed and compacted graph shows the unlikely slight decrease in compressive strength versus time relationship.



COMPRESSIVE STRENGTH VS TIME (DAYS)
ALL SAMPLES
COMPRESSIVE STRENGTH (PSI)
(Thousands)
0
1
2
3
4
5
0
2
4
6
8
DAYS
□ Field Mix, Lab Compact
—— Lab Mix & Compact
◇ Field Mix & Compact

CHAPTER 6
SENSITIVITY ANALYSIS

6.1 Introduction

An analysis was done on the following relationships:

1. The effect of variation of Moist Rodded Unit Weight (MRUW) on the ultimate layer thicknesses.

2. The effect of variation of the total water application on the water to cement (W/C) ratio and moisture content.

6.2 MRUW Values to Layer Thickness Relationship

The coarse aggregate moist rodded unit weight was varied by +/- 10%. Taking into account the effect of the angle of repose, which causes the trapezoidal shape of the layers, the change in coarse aggregate height changes the width of the top of the coarse aggregate layer that the fine aggregate rests on (coarse is on bottom, followed by the cement layer and then the sand layer). The change in thickness of each layer along with the overall thickness of the combined layers is tabulated in Table 6.1.

Table 6.1 Coarse Aggregate MRUW Sensitivity Analysis

Percent Change	+10%	+5%	0%	-5%	-10%
Coarse MRUW (pcf)	100.54	95.97	91.40	86.83	82.26
Coarse Agg. (in)	4.7	5.0	5.3	5.7	6.1
Cement (in)	0.6	0.6	0.6	0.6	0.6
Fine Agg. (in)	6.0	6.2	6.4	6.7	7.1
Total Thick (in)	11.3	11.7	12.3	13.0	13.8

Water does not add to angle of repose layer thickness since water permeates into the voids. The effect on the cement thickness was neglected due to the thinness of the layer (0.6 inches).

The change in total thickness of the layers for a 10% increase in the coarse MRUW amounts to a 1-inch decrease (8%), while a 10% decrease in the coarse MRUW amounts to a 1.5-inch increase (12%) in total thickness. These changes in volume of material input are not significant as discussed in the ACI Code (38).

The fine aggregate moist rodded unit weight was varied by +/- 10%. The effect of the angle of repose was taken into account. No effect occurs in the coarse aggregate due to the placement of the coarse layer on the bottom. The change in thickness of each layer along with the overall thickness of the combined layers is tabulated in Table 6.2.

Table 6.2 Fine Aggregate MRUW Sensitivity Analysis

Percent Change	+10%	+5%	0%	-5%	-10%
Fine MRUW (pcf)	94.33	90.04	85.75	81.46	77.18
Coarse Agg. (in)	5.3	5.3	5.3	5.3	5.3
Cement (in)	0.6	0.6	0.6	0.6	0.6
Fine Agg. (in)	5.7	6.0	6.4	6.9	7.4
Water	0.8	0.8	0.8	0.8	0.8
Total Thick (in)	11.6	11.9	12.3	12.7	13.3

As before, water had no effect and the cement-layer change was neglected. The change in total thickness of the

layers for a 10% increase in the coarse MRUW amounts to a 0.7-inch decrease (6%), while a 10% decrease in the coarse MRUW amounts to 1-inch increase (8%) in total thickness. These changes in volume of material input are not significant as discussed in the ACI Code (39).

6.3 Total Water Application to W/C Ratio and Moisture Content Relationships

The total amount of water added is controlled by the flowrate of the water and the speed of the watertruck. The flowrate of water and the watertruck speed were varied by +/- 10% to gauge the effect on the water to cement (W/C) ratio and overall moisture content. Table 6.3 tabulates the effects of flowrate variation.

Table 6.3 Flowrate Sensitivity Analysis

Percent Change	+10%	+5%	0%	-5%	-10%
Flowrate (gal/sec)	2.14	2.04	1.94	1.84	1.75
Volume of Water Placed (gal)	236	225	215	204	193
Water/Cement Ratio	54.4%	51.7%	48.9%	46.1%	43.4%
Moisture Content	5.5%	5.2%	4.9%	4.7%	4.4%

Table 6.4 tabulates the effects of watertruck speed variation.

Table 6.4 Watertruck Speed Sensitivity Analysis

Percent Change	+10%	+5%	0%	-5%	-10%
Velocity (ft/sec)	1.79	1.71	1.63	1.55	1.46
Volume of Water Placed (gal)	195	204	215	226	239
Water/Cement Ratio	43.9%	46.3%	48.9%	51.8%	55.0%
Moisture Content	4.5%	4.7%	4.9%	5.2%	5.5%

The range for the water to cement ratio is 36% to 60%. The low end of 36% is the lowest W/C ratio that typically permits full hydration of all cement powder (40). The high end of 60% is the maximum W/C ratio allowed according to ACI Code (41). The variations on the W/C ratios clearly fall within the acceptable range.

The maximum moisture content recommended is approximately 6% (42,43). The controlling factor is the required stiffness for compactive efforts by vibratory roller.

CHAPTER 7
CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

The CPM diagram details a construction sequence that takes approximately 2.25 hours to complete. The pavement may be used at +2.25 hours, however, rutting of approximately 1.5- to 2-inches will occur and should be rolled out within 30 minutes after occurring. If at least one hour of curing time is allowed after vibratory roller compaction, no settlement will occur due to a 400 psi stress which is the maximum wheel stress anticipated. Therefore, the pavement would be fully usable after approximately 3.25 hours from the beginning of construction. It is concluded that most of the strength of the concrete at the early time of +3.25 hours is due to the strength of the aggregates acting as a base course with minimal additional strength from concrete set. Therefore, compaction of the subgrade below the aggregates is required to provide additional resistance against settling. Further study should be completed to determine the long-term effect of such early use on the durability of the concrete after set. In an emergency situation over several days, the short-term use is unlikely to be hindered.

It is concluded that this method is superior to existing methods of runway repair since it requires no maintenance once placed (disadvanatge of crushed stone), provides a smooth interface with existing pavement (disadvantage of AM-2 matting), resists differential

settlement (pre-cast concrete slabs), and can rapidly produce large amounts of zero slump concrete (disadvantage of Cretemobile).

The mix design was capable of producing strengths of 3000 psi and above within 24 hours when properly proportioned, mixed, and compacted. This was evidenced by compressive tests conducted at 24 hours after mix preparation.

One of the most influential factors in this method is judged to be the degree of mixture of the concrete materials. Establishing reliable accurate mixing performance criteria would require extensive testing and analysis, therefore this performance criteria was not measured in the field test due to the limit of resources, time, and effort of personnel performing the field test. However, it is obvious that if the layers were not mixed at all, no strength would develop in the slab. Conversely, perfectly mixed material possesses the potential of attaining the anticipated mix design strength. The degree of mixing actually performed lies somewhere in between the two extremes with some irregularity in certain areas. Irregular mixing could cause an excess of concrete powder in one area possibly producing a very high strength sample and a lack of cement powder in another area producing a very weak sample.

Results from lab mixed and lab compacted test specimens are conclusive showing an increase in unit weight causes an

increase in compressive strength. The compressive strength increased over time as expected from the 1 day to 8 day compression tests. The unit weights were well-distributed producing similar average unit weights from the one-day test to the eight-day test.

Results from the field mixed and lab compacted samples were conclusive for compressive strength versus time relationships, but inconclusive for compaction to compressive strength relationships. The unit weights were clustered together (around 0.8% spread) producing similar average weights from the 14-hour test to the 24- and 72-hour tests. The clustering of unit weights also produced a clustering of compressive strengths (+/- 100 psi) for the 14-hour compression test. The slight decrease in compressive strength as compaction increased is concluded to be erroneous. It is concluded that the small decrease in compressive strength was caused by irregular mixing. The 24-hour test showed an increase in compressive for a small increase in compaction. Therefore, it is concluded that the 24-hour test was affected by other factors such as improper mixing which caused concentrations of cement powder producing higher and lower strength samples. The compressive strengths increased over time as expected.

The results from the field mixed and lab compacted were conclusive in showing a compressive strength increase versus increasing compaction. However, the compressive strength versus time results indicated an erroneous trend of slightly

decreasing strength over time. This result is concluded to be due to the high strength of 3550 psi for the 12C sample which raised the three-day test average strength to 2610 psi vice the expected value of approximately 2200 psi. The high strength is concluded to be due to possible improper mixing providing excess cement powder. The 12C sample also had the highest degree of compaction for the field mixed and compacted tests, thereby raising the compressive strength. The seven-day average compressive strength was 2440 psi creating an apparent 6.5% decrease in strength from the three-day test average of 2610 psi. The expected result was an increase of approximately 9% in strength from the three- to seven-day tests. Therefore, the total deviation from the normal was approximately 15%.

It is concluded that variations of the moist rodded unit weights of +/- 10% will not significantly affect the strength of the final mix.

A variation of +/- 10% in the flowrate or watertruck speed will also not significantly affect the final strength of the mix. The 10% increase in water applied will also not significantly affect the stiffness of the final mix, thereby allowing the use of vibratory equipment.

7.2 Recommendations

For future testing of this procedure, it is recommended that dry specific gravity tests and absorption tests be performed on the fine and coarse aggregates in accordance

with ASTM standards (ASTM C127 & C128) to use for the volume of solids calculation. This will allow a closer check (than the void ratio check) on the following:

1) Moist rodded unit weight (MRUW) calculation by using the MRUW value to calculate the weight of solids (Ws) and using the dry specific gravity to calculate a volume of solids (Vs).

2) Use the absorption parameter obtained from in-house testing and calculate a saturated surface dry specific gravity (SSDSG).

3) Check the values calculated for SSDSG against the SSDSG parameters obtained from the aggregate company's published laboratory results. Also check the absorption parameters obtained by in-house testing against aggregate company's published laboratory results. The calculated values should check within 1% or other appropriate amount as determined by sensitivity analysis.

The layer mixing method relies on the use of zero slump, roller compacted concrete. The zero slump concrete could be made quicker for small quantities by using a small portable mixer (16S mixer). The breakeven quantity (+/- cy) at which the layer mixing method surpasses the small mixer in production needs to be found. This will allow a selection of methods based on the known quantity of concete needed.

A criteria for degrees of mixing could be developed. Using the degrees of mixing, test specimens could be

prepared (four samples for each degree of mixing). Strength versus degrees of mixing relationships could be developed from test results. The degrees of mixing should be delineated as to mixing of aggregates with cement powder and mixing of mixed aggregates and cement powder with water. To be useful, descriptive definitions of appearance of the mix for each degree of mixture must be developed so that field personnel can competently relate the appearance of the mix in the field to a particular degree of mixture.

The field mix and compaction combination could be tested in a manner to insure a broad range of unit weights for each time interval tested. This will allow a more controlled comparison of unit weight versus compressive strength analysis. The test slab should be approximately five ft wide and ten roller widths long. The vibratory roller could then be used to compact each roller width length of the slab at different numbers of passes thereby creating different unit weights. The vibratory roller could start from one end traveling across the width of the slab for one pass down to the far end of the slab where ten passes across the width of the slab would be made. Core samples could then be cut from each section. A minimum of five laers should be cut from each section to provide sufficient samples to allow for sample deviations. The samples could be tested for compressive strength. Beams could also be cut and tested for flexural strength.

Since the slab will most likely be used within the

first few hours after construction, the rupture strength of the concrete will be minimal, therefore cracking of slab will occur. To further analyze the crack formation and ensuing settlement requires a finite element analysis. The finite element analysis should be done for different aircraft wheel loads (C-5, C-141, F-14, etc..) and slab configurations. The research goal would be to determine the degree of differential settlement that could occur in certain loading situations.

The use of a paver could also be used to lay down the aggregates using the external equivalent hole concept. For a large number of crater repairs, the paver may become very beneficial in speeding up layer placement.

APPENDIX (A)
FLOW CONTAINER CALCULATIONS FOR TRAPEZOIDAL CONTAINER

Minimum Width = 23 in   Minimum Length = 33 in

Maximum Width = 30.5 in   Maximum Length = 58.5 in

Slope of Width Sides = 0.625

Slope of Length Sides = 2.125

Maximum Depth = 12 in

Actual Depth = 10 in

Maximum Volume = 8.50 cf

Actual Maximum Width = 29.25 in

Actual Maximum Length = 54.25 in

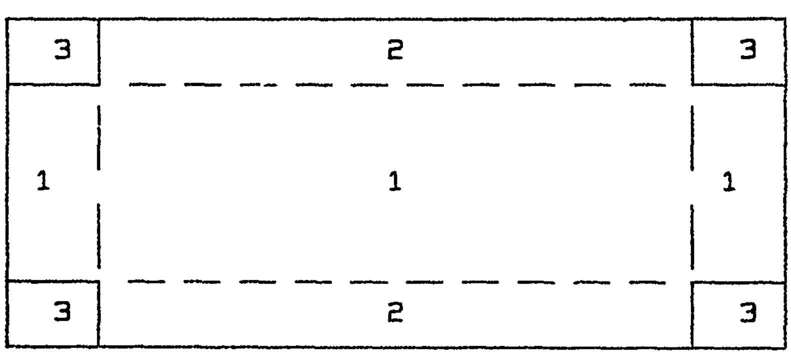
Trapezoidal Container is broken down into 3 sections.

Section 1 consists of the rectangular box having a length equal to the full length of the container, width equal to the minimum width, and depth equal to water depth.

Section 2 consists of the two triangular containers (i.e. half rectangular containers) having a length equal to the minimum length of the container, base width equal to the half of the difference between the maximum width minus the minimum width, and depth equal to the depth of the water.

Section 3 consists of the four remaining corners

The section divisions are illustrated below:



Section 1 = 4.989 cf
Section 2 = 1.414 cf
Section 3 = 0.3843 cf

Total = 6.787 cf

ActVolume   50.78 gal

APPENDIX (B)
CALCIUM CHLORIDE APPLICATION

Assumes all Calcium Chloride is placed.

Calcium Chloride Content = 32 lbs

Total Driving Time = 111 sec

Application Rate = 0.293 lbs/sec

Total Volume of Dilution Water = 10 gal

Concentration Calcium Chloride = 3.242 lbs/gal

Calc'd Application Rate = 0.090 gals/sec

Calc'd Application Rate = 11.06 secs/gal

Calcium Chloride Placed by Weight = 1.00%

APPENDIX (C)
MATERIAL REQUIREMENTS

Actual Crater Volume = 8.3 Cy

Volumes in Place	1 Cy	8.3 Cy
Coarse Aggregate *	0.751 Cy	6.256 Cy
Fine Aggregate *	0.599 Cy	4.989 Cy

* Includes FLUFF of aggregates

BASE COARSE OF COARSE AGGREGATE = 2 in

Extra Coarse Aggregate Required = 1.67 cy

Bags of TYPE III Cement Req'd = 34 bags
Add extra bags to prevent running short.

MATERIAL SUMMARY

	Required (cy)	On Hand (cy)	Reserve (cy)
Coarse Agg.	7.92	10.00	2.08
Fine Agg.	4.99	10.00	5.01
* Cement (bags)	34.49	50.00	15.51

APPENDIX (D)
UNIVERSAL ENGINEERING TEST RESULTS

UNIVERSAL
ENGINEERING SCIENCES
Consultants In Geotechnical Engineering • Environmental Sciences • Construction Materials Testing

Offices In
• Orlando
• Gainesville
• Fort Myers
• Merritt Island
• Daytona Beach
• West Palm Beach

February 22, 1990

Kaco Contracting Company
1911 Silver Star Road
Orlando, Florida 32804

Attention: Mr. Richard Coble

Reference: Roller Compacted Concrete Test Strip
1911 Silver Star Road
Orlando, Florida
Project Number: 15281-001-01CMT
Report Number: 2712 Reissued: 03-28-90

Dear Mr. Coble:

As requested, on February 12, 1990, Universal Engineering Sciences (UES) obtained six (6) additional cores test specimens from the roller compacted concrete test strip at the above noted site to be cut, capped, and tested for weight, volume, and compressive strength. The cylinders were also tested for weight, volume, and compressive strength yielding a comparison between the cylinders and field cores concrete.

The cores were obtained on February 12, 1990, utilizing a rotary drill diamond core bit and test for compressive strength on the following dates:

3 day test on 2/12/90,
7 day test on 2/16/90,
28 day test on 3/9/90.

All pertinent core test data is provided in tabular form on the accompanying sheet titled "Report on Compressive Strength on Hardened Concrete." The data pertinent to the field compacted cylindrical concrete specimen is as follows:

3532 Maggie Blvd. • Orlando, FL 32811 • (407) 423-0504

Cylinder No.	Age	Volume (ft3)	Unit Weight (pcf)	Before Cap Height (in.)	After Cap Height (in.)	Before Cap Weight (lbs.)	After Cap Weight (lbs.)	Diameter (in.)	L/D	Correction Factor	Total Load (lbs.)	Compressive Strength	Corrected Compressive Strength
1	14 hours	0.1292	141.64	7.877	8.267	18.30	19.44	6.00	1.38	0.9445	42,500	1500	1420
2	14 hours	0.1288	141.15	7.875	8.261	18.18	19.34	6.00	1.38	0.9445	48,250	1710	1610
3	24 hours	0.1490	140.53	9.108	9.380	20.94	21.76	6.00	1.56	0.9670	42,000	1490	1440
4	24 hours	0.1450	141.21	8.865	9.241	20.48	21.38	6.00	1.54	0.9660	77,250	2730	2640
5	3 days	0.1525	140.09	9.320	9.615	21.36	22.28	6.00	1.60	0.970	79,500	2810	2720
6	14 hours	0.1389	141.72	8.488	8.817	19.68	20.69	6.00	1.47	0.9560	41,500	1470	1410
7	14 hours	0.1425	140.53	8.708	9.016	20.02	21.02	6.00	1.50	0.960	47,750	1690	1620

UNIVERSAL ENGINEERING SCIENCES

REPORT ON COMPRESSSIVE STRENGTH OF HARDENED CONCRETE

Order No.: 15281-001-01CMT

Report No.: 2712

Date: 2-22-90

Core No.	LOCATION	Volume (ft^3)	Unit Wt. (pcf)	Age (days)	Total Length of Core Before	After	Area of Core (sq. in.)	Diameter (in.)	L/D	Correction Factor	Total Load (lbs)	Compressive Strength	Corrected Compressive Strength
1	3' N. & 3'E. of SW Corner	0.0193	140.56	7	5.5	5.54	6.03	2.77	2.00	-	15,000	2490	-
2	4' N. & 5' W. of SE Corner	N/S	N/S	28	6.0	5.6	6.03	2.77	2.02	-	23,700	3,930	
3	15' N. & 5' E. of SW Corner	0.0199	141.32	3	7.0	5.71	6.03	2.77	2.06	-	21,400	3550	-
4	14' N. & 5' W. of SE Corner	N/S	N/S	28	6.0	5.8	6.03	2.77	2.09	-	21,000	3,480	
5	3' S. & 4' E. of NW Corner	0.0195	138.67	7	8.25	5.60	6.03	2.77	2.02	-	14,400	2390	-
6	5' S. & 6' W. of NE Corner	0.0194	133.65	3	6.50	5.57	6.03	2.77	2.01	-	10,000	1660	-

NOTE A: Core No. 3 & 6 Tested on 2-12-90
Core No. 1 & 5 Tested on 2-16-90
Core No. 2 & 4 Tested on 3-09-90

NOTE B: Core Nos. 1, 3, 5, and 6 were not weighed prior to compressive strength test.

In review of the data on the test strip cores and the compacted concrete cylinder, the compacted concrete cylinders indicate similar compactive effort on each sample even though the height was varied. The unit weights of the cores and cylinder were similar, yet the compressive strength varied greatly.

We trust the presented information satisfies your immediate needs. However, if you should have any questions, please feel free to call.

Respectfully submitted,

Mark Naughton
Laboratory Supervisor
Universal Engineering Sciences

Fred J. Schmalzer, P.E.
CMT - Engineering Manager
Universal Engineering Sciences

MN:FJS:rls

cc: Client (2)
Paul Soares (1)
MN, UES (1)

APPENDIX (E)
CONCRETE TEST RESULTS

Table E.1 Compressive Strength Data Sheet

Cyl.	Diam. (in)	Ht. (in)	Vol. (cf)	Wt. (lbs)	Unit Wt. (pcf)	Comp. (%)
1C	4	6.325	0.0460	6.319	137.4	94.6%
2C	4	5.975	0.0435	6.263	144.1	99.3%
3C	4	6.325	0.0460	6.347	138.0	95.0%
4C	4	6.475	0.0471	6.715	142.6	98.2%
5C	6	7.894	0.1292	18.30	141.7	97.6%
6C	6	8.488	0.1389	19.68	141.7	97.6%
7C	6	7.875	0.1289	18.18	141.1	97.2%
8C	6	8.708	0.1425	20.02	140.5	96.8%
9C	6	9.108	0.1490	20.94	140.5	96.8%
10C	6	8.865	0.1451	20.48	141.2	97.2%
11C	6	9.320	0.1525	21.36	140.1	96.5%
Core						
12C	2.77	7.000	0.0244	3.450	141.3	97.3%
13C	2.77	6.500	0.0227	3.030	133.6	92.0%
14C	2.77	5.500	0.0192	2.696	140.5	96.8%
15C	2.77	8.250	0.0288	3.990	138.6	95.5%

Cylinder	Load (lbs)	Area (in^2)	Init. Comp. Stress (psi)	Red. Factor (%)	Corr. Comp. Stress (psi)
1C	39490	12.566	3140	0.9600	3010
2C	44650	12.566	3550	0.9300	3300
3C	51230	12.566	4080	0.9600	3920
4C	65010	12.566	5170	0.9600	4960
5C	42500	28.274	1500	0.9445	1420
6C	41500	28.274	1470	0.9560	1410
7C	48250	28.274	1710	0.9445	1620
8C	47750	28.274	1690	0.9600	1620
9C	42000	28.274	1490	0.9670	1440
10C	77250	28.274	2730	0.9660	2640
11C	79500	28.274	2810	0.9700	2730
Core					
12C	21400	6.026	3550	1.0000	3550
13C	10000	6.026	1660	1.0000	1660
14C	15000	6.026	2490	1.0000	2490
15C	14400	6.026	2390	1.0000	2390

APPENDIX F
COMPRESSIVE STRENGTH GRAPH DATA

TABLE (F1) - Graph Summary of Calculated and Actual Strengths

PRIMARY SORT ON CURING TIME,
SECONDARY SORT ON COMPACTION

SPEC.	COMP	FLEX	TENS	COMPACT'N (%)	TIME (HRS)	Y REV.	AVGS.
1C	3010	452	301	94.6%	24	2973	3820
1T	3546	567	355	96.1%	24	3409	97.5%
2C	3300	495	330	99.3%	24	4336	
1B	5405	811	507	100.0%	24	4555	
3C	3920	588	392	95.0%	192	3066	4760
2T	2435	390	243	96.0%	192	3763	97.3%
2B	5576	836	523	97.4%	192	4828	
4C	4960	744	496	98.2%	192	5408	
3B	6914	1037	648	100.0%	192	6739	
8C	1620	243	162	96.8%	14	1614	1520
7C	1620	243	162	97.2%	14	1507	
5C	1420	213	142	97.6%	14	1398	
6C	1410	212	141	97.6%	14	1394	
9C	1440	216	144	96.8%	24		2040
10C	2640	396	264	97.2%	24		
11C	2730	410	273	96.5%	72		2730
13C	1660	249	166	92.0%	72		2610
12C	3550	533	355	97.3%	72		
15C	2390	359	239	95.5%	168		2440
14C	2490	374	249	96.8%	168		

AVERAGE CORE COMPACTION = 96.4%
(FOR 3 & 7 DAY BREAKS)

Table (F2) Laboratory Mixed and Compacted Graph Data

COMP% CYLINDER DATALABEL	IND. COMP% B-RANGE	X-AXIS	AVG COMP% A-RANGE	ACTUAL COMPRESSIVE STRENGTHS
		0	97.4	
1C	94.6	1	97.4	
1T	96.1	2	97.4	
2C	99.3	3	97.4	
1B	100.0	4	97.4	
3C	95.0	5	97.4	3920
2T	96.0	6	97.4	2435
2B	97.4	7	97.4	5576
4C	98.2	8	97.4	4960
3B	100.0	9	97.4	6914
		10	97.4	

Table (F3) Laboratory Mixed and Compacted Graph Data

AVG COMPRESS STRENGTH	TIME (DAYS)
3820	1
4760	8

Table (F4) Field Mixed and Lab Compacted Graph Data

CYLINDER DATALABEL	IND. COMP%	X-AXIS	AVG COMP%
		0	97.1
8C	96.8	1	97.1
7C	97.2	2	97.1
5C	97.6	3	97.1
6C	97.6	4	97.1
9C	96.8	5	97.1
10C	97.2	6	97.1
11C	96.5	7	97.1
		8	97.1

Table (F5) Field Mixed and Lab Compacted Graph Data

AVG COMPRESS STRENGTH	TIME (DAYS)
1520	0.6
2040	1.0
2730	3.0

Table (F6) Field Mixed and Compacted Graph Data

CYLINDER DATALABEL	IND. COMP%	X-AXIS B-RANGE	AVG COMP%	ACTUAL COMPRESSIVE STRENGTH
		0	96.4	
12C	97.3	1	96.4	3550
13C	92.0	2	96.4	1660
14C	96.8	3	96.4	2490
15C	95.5	4	96.4	2390
		5	96.4	

Table (F7) Field Mixed and Compacted Graph Data

AVG COMPRESS STRENGTH	TIME (DAYS)
2610	3
2440	7

Table (F8) Average Compressive Strengths

FLCOMPDA - Field Mixed, Lab Compacted
LLCOMPDA - Lab Mixed, Lab Compacted
FFCOMPDA - Field Mixed, Field Compacted

DAYS	FLCOMPDA	LLCOMPDA	FFCOMPDA
0.6	1520		
1.0	2040	3820	
3.0	2730	4090	2610
7.0		4630	2440
8.0		4760	

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